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A Member shall be an officer or employee of a public or private water works, an engineer, chemist or other person qualified to aid or interested in the advancement of knowledge relative to water works.

An Honorary Member shall be a person of acknowledged eminence in some branch of water supply or of engineering.

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A Corporate Member shall be either a Water Board, Commission, Company or Municipal Corporation.

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This Association has six regular meetings each year, all of which, except the annual convention in September, are held at Boston.

Table of Contents.

PAGE

Portrait of President David A. Heffernan ..... Frontispiece

Determining the Mean Precipitation on a Drainage Basin. By Robert E. Horton 1

Copper Sulphate Treatment of Cape Pond, the Source of Water Supply for Rockport, Mass. By E. Sherman Chase 48

Latest Developments in Chlorine Control Apparatus for Sterilization of Water Supplies. By Gilbert H. Pratt 63

Standardization of Fire Hose Threads. By Major J. Hastings Howland 73

Cast-Iron Service Pipe. By J. R. McWane 79

Further Studies of Water Purification by Filters Charged with Aluminum or Ferric Hydroxide. By H. W. Clark 84

Proceedings:

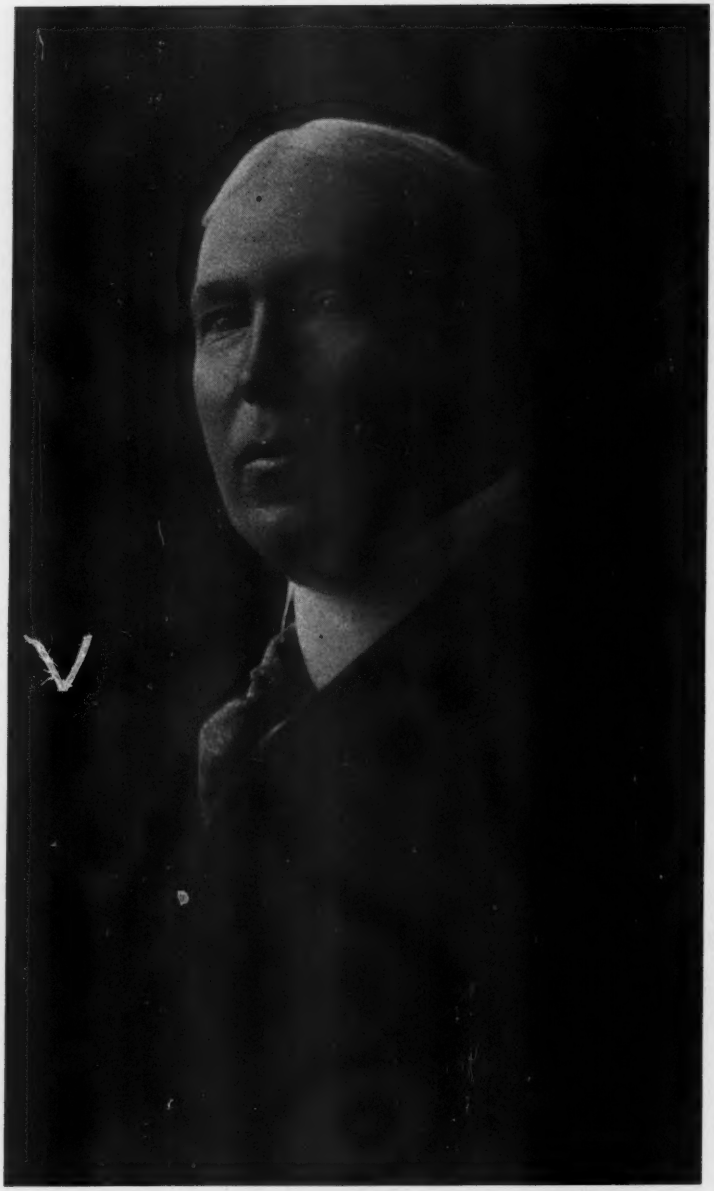
December Meeting 94

January Meeting 94

Obituary: Lewis M. Bancroft 96

Louis Henry Knapp 97

Hiram Francis Mills 98



DAVID A. HEFFERNAN.

President of the New England Water Works Association.

1923–1924.

New England Water Works Association

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This Association, as a body, is not responsible for the statements or opinions of any of its members.

DETERMINING THE MEAN PRECIPITATION ON A DRAINAGE BASIN.

BY ROBERT E. HORTON.*

(A summary of this paper was read on January 8, 1924. Members are requested to submit written discussions to the Editor.)

INTRODUCTION.

In a previous paper (ref. 1) the author described the usual methods of measurement of rainfall and snow. In the present paper some supplemental data are given, and the studies are extended to cover the application of rainfall data to the determination of the mean precipitation on a drainage basin, month by month, or season by season. Methods of determining the mean rainfall over a drainage basin apparently have received less attention than they deserve. The matter is neither so simple that it does not require scientific study, nor are the data so variable that no general rules may be laid down.

At the Holyoke meeting of this Association in September, 1920, a Committee on Water Losses from Drainage Basins was appointed. In developing a program for that committee, it became evident that to secure reliable estimates of water losses from drainage basins great care must be exercised in estimates of the mean precipitation over the various areas involved.

It fell to the author as Chairman of the Committee on Water Losses to undertake a detailed study of methods of estimating the mean precipitation over drainage basins. From this study standard methods for use by the Committee were formulated. It seemed worth while to publish the details of various methods and corrections, some of which are new, and this paper is presented as a contribution from the work of the Committee on Water Losses. The object in view has been to devise means of making the best possible or, in other words, the most probable determination of the monthly, seasonal, annual, or mean precipitation over a drainage basin, with a minimum of labor.

*Consulting Engineer, Albany, N. Y.

It will perhaps be a surprise to some that so many calculations and corrections are required. There may be those who will look askance at the procedure outlined, preferring a "straight uncorrected average" as being all that is justified by the nature and accuracy of the data usually available. If there be those who think that correction of rainfall records for deficiency of catch of snowfall, variations in elevation and the like, are "too theoretical," they should not overlook the fact that in the analysis of run-off and water loss data consistent results can often be obtained when these corrections are made in cases where the uncorrected rainfall data lead to absurdities.

The word "theoretical" when used as an apology is sometimes a covert synonym for "laborious." There is no pedantry in the use of theoretical corrections where experience indicates that such corrections are necessary. Their omission is in the same class with the neglect of impact and shearing stresses in the design of a railroad bridge, or the omission of ice pressure and upward pressure underneath the base of a dam, where such stresses will actually occur. The author believes that the methods here given are to be classified as "practical" rather than "theoretical," because the necessity for them has arisen from actual experience, and because the word "practical" in its best sense means that happy combination of experience and theory which most nearly fits actual conditions.

Collation of Rainfall Records.

The first step is to prepare a good outline tracing or base map showing the boundaries of the drainage basin, with marginal room sufficient to show the locations of rainfall stations around as well as within the area. Indicate on the map the locations of all rainfall stations within or closely adjacent to the drainage basin for which records are available covering all or some portion of the required period. Fig. 1 is such a base map.

Then make up a check sheet similar to Fig. 2, showing graphically the period covered by each precipitation record. From a study of this sheet, in conjunction with the base map, determine whether or not rainfall records complete for the whole required period are available in sufficient number and so distributed as to give a good basis of computation of the average precipitation over the area. If so, fragmentary records covering only a small part of the required period may sometimes be disregarded. If there are large gaps in which are insufficient complete precipitation records, but within which fragmentary records are available, the determination of the rainfall should usually be made for two or more subperiods, the subdivisions being made in such a manner as to obtain as complete a series as possible of well-distributed rainfall records over the whole area in each subperiod.

The monthly, annual, and seasonal precipitation at each station should next be compiled. For various reasons it is desirable to determine sepa-

rately the precipitation at each station for the winter six months' period, November to April inclusive, and for the summer six months' period, May to October inclusive. Having compiled the data the matter of making

FIG. 1.— RAINFALL STATIONS, SHENANDOAH RIVER BASIN, VA.

certain corrections which may be necessary should next be taken up, together with a critical examination of all the records with a view to eliminating any which may be erroneous.

Interpolation of Missing Records.

In most of the states there are few long rainfall records which are entirely complete. Various methods of interpolation or filling out the missing monthly values may be used. The results of an extended study of the relative accuracy of different methods or interpolation of missing monthly rainfall amounts has recently been published (ref. 2). As the results of that study are available in the *Monthly Weather Review* they need not be repeated here further than to outline certain of the methods

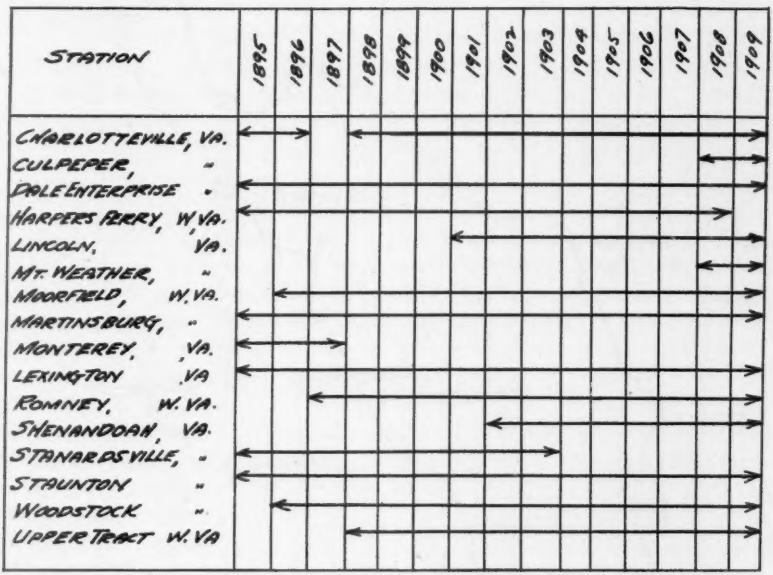

Fig. 2.— Check List of Rainfall Records, Shenandoah River Drainage Basin Above Millville, W. Va.

of interpolation which were found to meet best the requirements of giving good results with reasonable labor.

Missing months in a rainfall record are most commonly due to:

(*a*) Absence or illness of the observer.

(*b*) Accidents to the gage or record.

(*c*) Changing of observers.

Interpolation methods are of two classes:

(1) Those requiring only contemporaneous records at surrounding stations;

(2) Those requiring long-term means or normals.

It is convenient to designate surrounding stations used in interpolation as "base stations," while the station for which the interpolation is made is designated the "interpolation station." It should be borne in mind

that rainfall records are likely to be used over and over, and the interpolations, if made in the most accurate and logical manner possible in the first instance, are of permanent value. In the case of small drainage basins the number of applicable rainfall records is usually small. In such cases, if errors of interpolation are large they may affect the accuracy of the final results. Methods of interpolation should therefore be used which will reduce the resulting errors to a minimum. The most reliable methods of interpolation of missing monthly rainfall amounts depend upon the principle established long ago by Kämtz, that the relative amounts of rainfall in different time intervals at two adjacent stations are usually about the same. In other words, if the rainfall at each station in a given month be

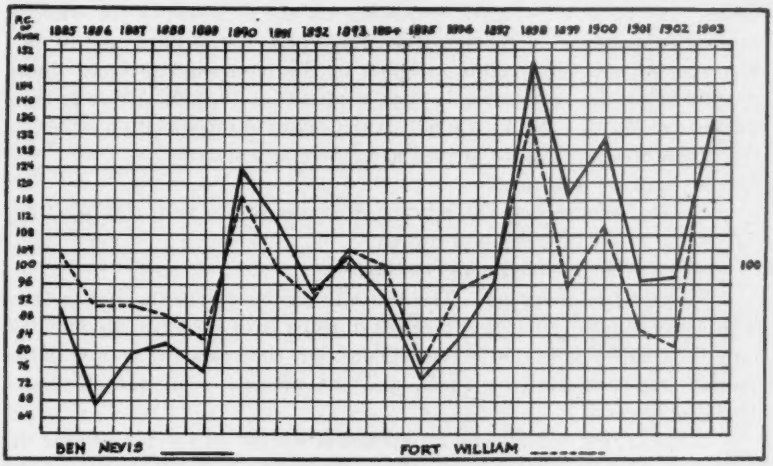

FIG. 3.—THE VARIATION OF RAINFALL AT BEN NEVIS AND FORT WILLIAM EXPRESSED AS PERCENTAGES OF THE RESPECTIVE AVERAGES.

expressed as a ratio either to the mean for the month or to the total for the year, these ratios will be nearly identical, although the actual amounts of rainfall may be widely different.

This principle is well illustrated by the rainfall at Ben Nevis and Fort William, Scotland (Fig. 3). Fort William is on the west coast of Scotland, the rain gage being 31 ft. above sea level. The Ben Nevis station is at the summit of the mountain, about 4 400 ft. higher elevation and 4 miles eastward inland. The mean precipitation at Ben Nevis is 160.78 in.; that at Fort William 78.58 in., or less than half as much. In spite of this wide difference in actual rainfall amounts it will be seen from Fig. 3 that the relative rainfall amounts in different years are nearly identical.

Mean of Three Surrounding Stations. This is probably the method of interpolation most commonly used. In the study referred to (ref. 2) it was found that the error of an interpolation based on three surrounding

stations is only about two-thirds as great as if the rainfall at a single adjacent station was substituted for the missing value. This method is subject to the defect that it gives equal weights to the base stations, although these may not be equidistant from the interpolation stations. The results by this method can be somewhat improved by giving the rainfall amounts for the three adjacent stations weights inversely proportional to their distances from the interpolation station.

Methods of Weighting Interpolated Values. In applying methods of interpolation where several surrounding stations are used, each station yields in general a value of the interpolated quantity. These may be given equal weights, in which case the adopted value of the interpolated quantity is the arithmetic mean of the several (usually three) values, or the individual values may be given weights, depending on the locations of the stations or their similarity. In general, when three base stations are used, if W_a, W_b, and W_c are the weights assigned to the interpolated values, these weights being chosen relative to such a scale that $W_a+W_b+W_c=1$, then $d=W_a\ d_a+W_b\ d_b+W_c\ d_c$, where d is the required interpolation value.

These weights may be arrived at by several methods:

(*a*) By judgment.

(*b*) Inversely as the relative distances of the base stations from the interpolation station.

(*c*) The inclined-plane method.

Weighting by Judgment. Theoretically this is perhaps the best method if properly applied, since it is possible in using it to take into account not only the relative positions of the stations, but the nature, if known, of the rainfall variation between them, and the differences in rainfall causes applying to each.

Consider, for example, an interpolation station on the plains near the foot of a mountain range, with three interpolation stations, two on the plains and one on the mountains. Now suppose the conditions are such that a large proportion of the rainfall on the plains is convective, while at the mountain station the rainfall is more largely orographic. Obviously the plains stations should be given greater weight than the mountain station, if all were equidistant from the interpolation station. Among disadvantages of weighting by judgment are:

(1) The factors affecting the proper weights to be applied are, except relative distances, in general unknown, or not quantitatively known; therefore,

(2) Different operators using the same data will not obtain the same results.

Weighing by Inverse Distances. If X_a, X_b, X_c are the distances of the base stations from the interpolation station, in any linear units, then if weights are assigned to the three stations each inversely proportional to its distance from the interpolation station and on such a scale that

$$W_a+W_b+W_c=1$$

the numerical values of the weights can be derived as follows:
Take reciprocals of X_a, X_b, and X_c. Let

$$\frac{1}{X_a}+\frac{1}{X_b}+\frac{1}{X_c}=M$$

then

$$W_a=\frac{1}{M X_a},\ W_b=\frac{1}{M X_b},\ W_c=\frac{1}{M X_c}.$$

These relative weights once determined can be applied to all interpolations involving a given group of stations.

Inclined-Plane Method. This method was devised by the author with a view to applying simultaneous or contemporaneous records in the

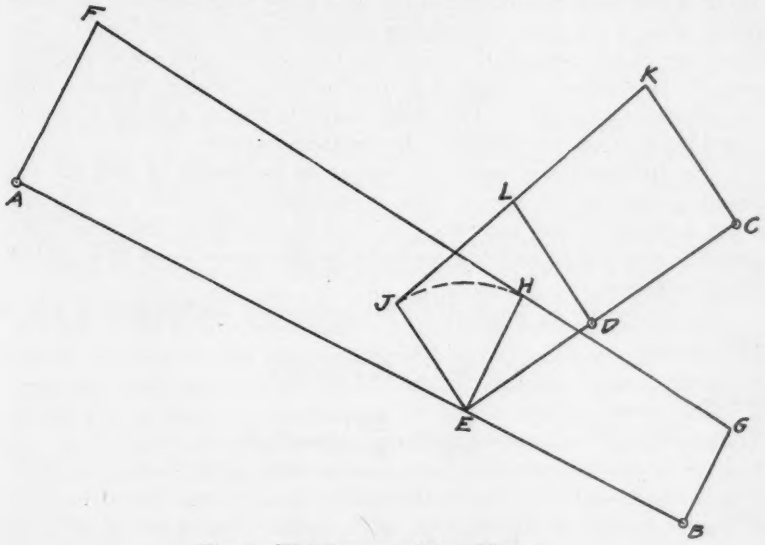

FIG. 4.—THE INCLINED-PLANE METHOD.

most logical manner possible, thus obtaining the best practicable result with the least expenditure of labor, since all methods dependent on simultaneous or contemporaneous records alone are much simpler of application than methods involving the use of long-term means. It depends upon the principle that the position of a plane is completely determined by the coördinates of three points in the plane. In the absence of information to the contrary, the best assumption which can be made is that rainfall varies uniformly between adjacent stations.

Select three base stations, A, B, C, Fig. 4, surrounding the interpolation station. On a suitable map showing the relative positions of the stations, connect any pair, A, B, of the base stations by a line, and erect perpendiculars to this line at the two stations, and measure off on each

perpendicular a length proportional to the precipitation at that station for the period to be interpolated. On the assumption of uniform variation, the precipitation at any point along this base line will be proportional to the ordinate from the base line to the line connecting the two plotted points. Draw another line from the third base station, C, through the interpolation station, D, intersecting the base line AB at E. Erect a perpendicular to AB at E, intersecting the line FG at H. The precipitation at E is assumed to be proportional to EH. Draw perpendiculars to CE, one at C proportional to the precipitation at that station, and one at E equal to EH. Connect these by a line JK. Then a perpendicular DL at the interpolation station will have a length proportional to the precipitation at D. The graphical construction is extremely simple. Only simultaneous records are used and the method involves but little more labor than the use of the direct mean of the three surrounding stations, but it is more logical. The direct use of the mean of three surrounding stations gives equal weight to each of the stations, although they may be widely different distances from the interpolation station. The weight given to remote stations in the inclined-plane method decreases as the distances increase.

The inclined-plane method of combining the results of data for surrounding stations can also be used in conjunction with the Fournie and other multiple station methods. It is to be considered, therefore, as a principle rather than as being restricted to the narrow limits of a method of interpolation.

It can readily be shown from the geometrical construction of Fig. 4 that

$$P_d = K_1 P_a + K_2 P_b + K_3 P_c$$

where

$$d_1 = \frac{A E}{A B},\ d_2 = \frac{E D}{E C},\ \text{on Fig. 4,}$$

and

$$K_1 = 1 - d_1 - d_2 + d_1 d_2$$
$$K_2 = d_1 - d_1 d_2,$$
$$K_3 = d_2;$$

also

$$K_1 + K_2 + K_3 = 1$$

or the sums of the coefficients is unity, providing an easy check on computations.

Fournie Method. This method has been extensively used for interpolation of missing rainfall years. It is equally applicable to the interpolation of missing months, but like all other methods, gives less accurate results for monthly than for annual interpolations, owing to the relatively greater variability of rainfall for short periods than for full years. To apply this method, three surrounding stations are selected for each of which there

is a rainfall record for a period of several years simultaneous with the rainfall record at the interpolation station. Calling means for these three stations A, B, and C, and calling the mean for the interpolation station D, the ratios $\frac{D}{A}$, $\frac{D}{B}$, and $\frac{D}{C}$ are worked out for the simultaneous periods covered by all four records. Calling the actual precipitations at the three base stations for the period for which interpolation is required a, b, and c, respectively, these values are multiplied by the corresponding ratios and the mean of the three products taken as the interpolated rainfall for the interpolation station. The method is laborious, especially in view of the fact that in many cases gaps will be found in the records for the base stations which must themselves be filled out before long-term ratios can be computed. Sometimes one of the base stations chosen is necessarily much more remote from the interpolation station than are the others. Fournie's method gives equal weight in the result to the values obtained from the different base stations, whereas it is likely that the precipitation for the given month at the interpolation station conforms more closely to that at a near-by station than to that at a remote station. Obviously the Fournie method can only be applied where records have been kept for a number of years. It does not depend on contemporaneous records and considerable research is often necessary to compile the data for computing the ratios of the means, even where the records for the base stations are complete.

The Fournie method utilizes data at the interpolation station. The inclined-plane method does not utilize data for the interpolation station. A combination of the inclined plane and Fournie methods serves to make use of the data for the base station and at the same time provides for weighting the results obtained at surrounding stations. The combination of the two methods is accomplished by first computing three values of the precipitation at the interpolation station by the Fournie method. Then instead of taking their average, the interpolated value to be used is determined by applying the inclined-plane method to these three results.

Use of Pluviometric Ratios. The precipitation at a given station in a given month expressed as a percentage of the total for the year has been designated by Angot the pluviometric coefficient. As indicated by Kämtz principle, these ratios are usually in good agreement for a given month at adjacent stations, although the actual precipitation amounts may differ. A similar ratio of a slightly different form may be used for interpolation purposes. This is the ratio of the precipitation in a given month to the total for the remaining eleven months of the same year. This may be called the pluviometric ratio. Expressed mathematically, let P_1, P_2, and P_3 be the rainfall amounts for the given month at the three reference stations. Let V_1, V_2, V_3, and V be the total precipitation for the remaining eleven months of the year at the three base stations and at the interpolation station, respectively. Then find the ratios $\frac{P_1}{V_1}$, $\frac{P_2}{V_2}$, $\frac{P_3}{V_3}$, and if P is the required

precipitation at the interpolation station,

$$P = V\frac{P_1}{V_1} = V\frac{P_2}{V_2} = V\frac{P_3}{V_3}.$$

These values in general will not be identical. The final interpolated rainfall may be determined either by taking the average of the three values, by weighting them in inverse proportion to the distances of the base stations from the interpolation stations, or by applying the inclined-plane method to these values. It will be seen that this method is identical in principle with the Fournie method, but by utilizing the pluviometric ratios the use of long-term normals is avoided and the labor of interpolation is in general greatly reduced without sacrifice of accuracy in the results.

TABLE 1.

AVERAGE ERROR IN PER CENT OF TRUE MONTHLY RAINFALL RESULTING FROM RAINFALL INTERPOLATIONS FOR 12 INDIVIDUAL MONTHS AT DIFFERENT STATIONS.

	Dannemora, N. Y.	New Iberia, La.	T. S. Ranch, Colo.	North Bloomfield, Cal.	Average of 4 Stations.
(1)	(2)	(3)	(4)	(5)	(6)
Average precipitation for months interpolated—inches	2.71	4.52	0.96	3.61	2.95
Mean arithmetic error of monthly interpolation—per cent of true monthly precipitation					
Nearest station	29.1	27.4	49.0	38.2	32.9
Mean of 3 surrounding stations	15.9	23.9	52.1	11.7	20.7
Inclined-plane method	20.3	18.4	42.8	20.5	21.4
Fournie method	15.9	23.7	45.9	24.6	24.1

Accuracy of Interpolation. There is, of course, no absolutely accurate or certain method of rainfall interpolation. The accuracy which may be attained in a given case depends not only on the method used, but also on the proximity of the base stations to the interpolation station and on the variability of rainfall in the region. The less the variability the greater the accuracy of interpolated values. Both the variability of rainfall and the percentage error by monthly interpolation are usually less for stations having high than for those of low mean annual rainfall. The degree of accuracy attainable in rainfall interpolation is illustrated by the data given in Table 1. Determination of the errors for the pluviometric ratio method in the form described above have not been made, but this method would apparently be equal to the better of the other methods in accuracy. The important points brought out by Table 1 are:

(1) Three-station methods give arithmetic errors only about two-thirds as great as single station methods.

(2) As between the various three-station methods described, the comparisons available are insufficient to determine their relative accuracy.

There is apparently little choice in this respect. The method used in a given case may be determined by considerations of the data available and labor involved. Other studies, which need not be described here, indicate, however, that the accuracy obtainable by a combination of either the Fournie or pluviometric ratio method with the inclined-plane method, or by weighting the results by the method of inverse distances, is somewhat greater than that obtainable by the use of either the Fournie or pluviometric ratio method alone. The labor is, however, increased when the combined methods are used.

Correction for Snow.

A not uncommon method of measuring winter precipitation consists in exposing the ordinary 8-in. rain-gage can with funnel and measuring tube removed. The snow caught in the gage can is then reduced to water and measured. Experience shows that owing to wind effects a rain-gage can so exposed almost invariably catches less than the amount of snow which falls on the level in a spot protected from drifting. Experience also shows that even where run-off records are of unquestionable accuracy, and certainly not excessive, still the run-off for the winter season, November to April, is often nearly equal to and sometimes greater than the measured precipitation for the same season. This occurs in spite of the fact that all available information indicates that there is an average monthly evaporation loss from snow surfaces ranging from $\frac{1}{2}$ to 1 in., and furthermore there is, as a rule, considerable contribution to, or increase of, surface and underground storage during the winter season. The conclusion is inevitable that measured precipitation for the winter season in northern states is quite generally deficient. Correct snow measurements can apparently be obtained only by:

(*a*) Ground or snow-board measurement; or

(*b*) The use of a gage protected by a Nipher shield.

Although an American invention, available since 1878, this valuable device has been little used in the United States. Ground measurements of snow are made at most of the 200 regular U. S. Weather Bureau stations and probably at many private rainfall stations maintained by water-works departments and manufacturing companies. At a large number, probably a majority, of the 4 000 coöperative rainfall stations in the United States, measurement of snow is made by exposing the gage can with the funnel removed. Ground measurement appears to be more common in high mountain and northern stations where snowfall is heavy than at southern stations. The use of the rain-gage can for snow measurement may lead to grossly deficient results, as illustrated by the following two

cases. These show the water equivalent of the snow caught in the rain gage can as compared with the actual water equivalent of the snowfall as determined by ground measurement in two heavy storms at Albany, N. Y.

	Dec. 26, 1913.	Dec. 24, 1923.
Caught	0.43 in.	0.28 in.
Fell	1.43 „	1.13 „
Correction factor	3.33	4.04

In the first storm the actual precipitation was 3.33 times and in the second storm 4.04 times the amount caught in the gage can. Wind action deflects

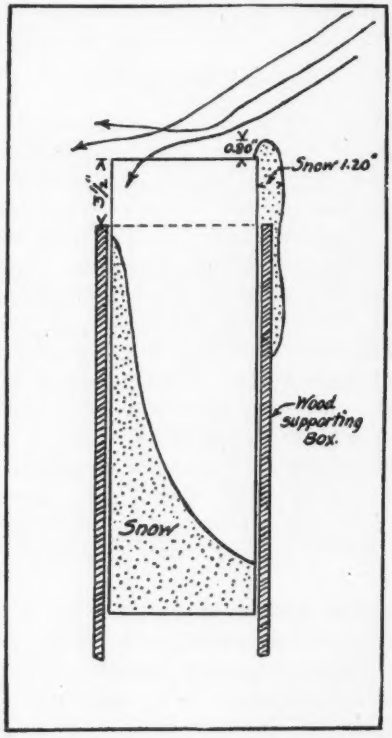

FIG. 5.— ACTION OF SNOW ON RAIN GAGE.

rain as well as snow from a rain gage, introducing an error of deficiency. The specific gravity of snow is much less than that of rain and the deficiency in catch for a given wind velocity is greatly increased for snowfall. The manner in which wind eddies deflect snow away from a rain gage is illustrated by Fig. 5. The error in catch of snowfall may be determined either by comparison of the gage-can measurement with ground measurement,

or by comparison of an unprotected gage with one having a Nipher shield. The results of a comparison by the latter method are given by Hann-Süring (ref. 3).

COMPARISON OF SHIELDED AND UNSHIELDED RAIN GAGES.

Excess of shielded Nipher gage over unshielded gage, per cent.	Wind Velocity. 2–3 mi. per hr.	12 mi. and over.
Dry snow	13 per cent	60 per cent
Moist snow	8 „ „	20 „ „
Light rain	8 „ „	10 „ „
Heavy rain	1 „ „	1 „ „

Hann-Süring have shown that many existing long rainfall records are defective due to this cause, especially in northern climates where the

TABLE 2.

COMPARISON OF SNOWFALL MEASURED BY STANDARD RAIN-GAGE OVERFLOW CAN AND BY SNOW BOARD.

HORTON HYDROLOGIC LABORATORY, 1920.

Date 1920.	Depth on Snow Board.	Water Equivalent of Snow.		Ratio Board. / Can.	Total Depth on Ground.
		Gage Can.	Snow Board.		
(1)	(2)	(3)	(4)	(5)	(6)
	In.	In.	In.		In.
Jan. 17	5.88	.142	.151	1.06	
„ 19	0.25	.053	.014	0.26	7.50
„ 20	1.00	.041	.041	1.00	8.50
„ 24	4.25	.209	.230	1.10	11.87
„ 25	1.50	.046	.057	1.24	13.00
„ 28	T	.0087	.003	0.34	11.50
Feb. 7	8.00	.544	.600	1.10	20.13
„ 12	0.75	.159	.193	1.21	18.37
„ 16	3.63	.175	.302	1.73	15.00
„ 20	0.19	.018	.018	1.00	
„ 22	0.75	.022	.029	1.32	
„ 24	2.38	.071	.091	1.28	
TOTAL		1.489	1.729	1.16	

snowfall is heavy. For example, at Peiszenburg it was found that the actual precipitation is 84 per cent greater than that given by an existing long record, while at Breslau a correction factor of 1.33 is required.

Having found it necessary to correct many precipitation records for deficiency of snow measurement before they could be used for water loss studies, a series of comparative measurements was instituted at the author's laboratory, latitude 42° 40′, elevation 300 ft. above tide. The results covering four winter seasons are given in Tables 2, 3, and 4.

The true snowfall was determined by measurements on a snow board devised by the author and consisting of a sheet of white beaver board about 16 in. square, with a layer of cotton flannel, nappe uppermost, tacked on

the surface of the beaver board. After each reading was taken the snow board was cleaned and dried and laid on the surface of the newly fallen, undisturbed snow. In taking the readings the gage can was first weighed, the snow then removed therefrom, and a sample cut out of the snow on the

TABLE 3.

COMPARISON OF SNOWFALL MEASURED BY STANDARD RAIN-GAGE OVERFLOW CAN AND BY SNOW BOARD.

HORTON HYDROLOGIC LABORATORY.

1920–21 AND 1921-22

Date.	Depth on Snow Board.	Water Equivalent of Snow Caught.		Ratio Board. / Can.	Density of Snow.
		Gage Can.	Snow Board.		
(1)	(2)	(3)	(4)	(5)	(6)
	In.	In.	In.		
1920					
Dec. 27	3.42	.578	.584	1.01	.171
1921					
Jan. 8	0.42	.082	.090	1.10	.214
„ 17	0.42	.025	.049	1.96	.117
„ 20	0.25	.032	.041	1.28	.164
„ 31	2.00	.287	.289	1.00	.144
Feb. 4	0.42	.007	.014	2.00	.033
„ 8	1.75	.186	.189	1.01	.108
„ 11	7.25	.322	.717	2.22	.099
„ 12	4.50	.314	.343	1.09	.074
„ 21	7.12	.768	.802	1.04	.113
„ 27	0.50	.017	.017	1.00	.034
TOTAL	28.05	2.618	3.135	1.20	.115
1921					
Nov. 26	2.00	.082	.157	1.93	.079
Dec. 6	0.06	.000	.001	∞	.016
„ 13	0.19	.081	.049	0.61	.388
„ 21	0.03	.003	.004	1.20	.133
„ 25	2.25	.280	.286	1.04	.127
1922					
Jan. 4		.110	.129	1.11	—
Feb. 10	0.38	.019	.020	1.05	.053
„ 13	0.50	.080	.085	1.05	.170
„ 19	0.75	.049	.069	1.40	.092
„ 27	2.62	.235	.254	1.08	.097
Mar. 2	5.50	.256	.284	1.11	.052
TOTAL	14.28	1.195	1.338	1.12	.085

snow board by inverting the gage can over the snow board, like a cookie cutter, then picking up gage can and snow board together, so as to get a perfect sample in the gage can. The gage can was then again weighed. An accurate torsion balance was used, making it possible in all cases to determine the water equivalent of the snow to the nearest thousandth of

an inch of water. It is convenient to note that using the U. S. Weather Bureau pattern 8-in. diameter rain gage, 1 oz. of snow caught therein is equivalent to a depth of 0.034 in. of water.

It will be noted that in very light snow flurries the amount caught on the snow board may be equal to or less than that caught in the gage can.

TABLE 4.

COMPARISON OF SNOW BOARD AND RAIN-GAGE CAN MEASUREMENTS OF SNOWFALL.

WINTER, 1922–23.

Date.	Depth on Snow Board.	Water Equivalent of Snow Caught. Inches.		Board Per Cent of Can.	Density of Snow.	Remarks.
		Gage Can.	Snow Board.			
(1)	(2)	(3)	(4)	(5)	(6)	(7)
	In.	In.	In.			
1922						
Dec. 15..	2.12	.235	.276	117.5	.130	
„ 18..	5.00	.647	.737	114.0	.147	
„ 29..	8.25	1.318	1.466	111.2	.178	
1923						
Jan. 4..	3.88	.384	.394	102.5	.102	
„ 7..	3.50	.307	.391	127.4	.112	
„ 8..	8.50	.782	.924	118.1	.109	
„ 15..	4.50	.462	.467	101.0	.104	
„ 29..	2.75	.138	.154	111.7	.056	
Feb. 7..	2.25	.146	.160	109.5	.071	
„ 10..	2.25	.145	.171	118.0	.076	
„ 13..	3.37	.436	.592	135.9	.176	
„ 14..	0.12	.032	.0086	26.8	.072	
„ 25..	1.75	.082	.178	217.5	.102	
Mar. 26..	0.25	.025	.0327	129.8	.130	
TOTAL.	48.49	5.139	5.952	116.%[a]	.123[b]	

Area of 8 in. gage can = 50.27□ in.

$$\frac{62.4 \times 16}{1728} = .5777 \text{ oz. per cu. in.}$$

$$50.27 \times .5777 = 29.041 \text{ oz. per 1 in. depth in can.}$$
$$= .0344 \text{ in. depth per 1 oz. in can.}$$

$$a = \frac{5.952}{5.139} = 116 \qquad b = \frac{5.952}{48.49} = .123$$

In all heavier snows the catch on the snow board was greater, and by a fairly consistent percentage. Much of the snow falls when the temperature is about 32 degrees, and while the type of snow board used was specially designed to simulate a snow surface, and prevent melting, the results indicate that in very light snow flurries the snow board may give deficient results.

During the first two winters the rain-gage can and the snow board were both exposed on the ice about 25 ft. from the north edge of a pond at the author's laboratory. The pond is about 100 ft. wide and several hundred feet in length, running in an easterly and westerly direction. The north slope to the pond ranges from 10 to 20 ft. per hundred and the pond is protected on the northern edge by trees and shrubs. The south bank is abrupt and wooded. Snow drifts on the pond surface only on rare occasions. The results were, however, sometimes lost by snow blowing off the snow board, which was accordingly removed to a point 50 ft. north of the pond for the third and fourth winters. The average correction factor to be applied to the gage-can measurement to obtain the true snowfall, expressed as water equivalent, was as follows:

1919–20	1.16
1920–21	1.20
1921–22	1.12
1922–23	1.16
Average of four seasons	1.16

The consistency of these results is gratifying and indicates that a fairly reliable correction for deficiency of catch of snowfall may be made where necessary. In order to apply these results elsewhere it is necessary to take into consideration, —

(1) The gage can used in the experiments had its funnel about 4 ft. above ground and had as good protection from wind action by surrounding adjacent slopes, trees and shrubs as can ordinarily be obtained. Only very well located rain gages will give as small a deficiency of snowfall catch. For the average voluntary rainfall station the snow correction should probably be greater.

(2) In the climate of Albany and vicinity two-thirds to three-fourths of the precipitation from November to April, inclusive, falls as snow.

It is usually desirable to make the correction once and for all to the entire winter precipitation. The percentage of the total precipitation falling as snow varies both with latitude and elevation. In extreme northern climates the snow is drier and the deficiency of catch increases. For the purpose of correcting winter precipitation in estimating water losses a tentative correction factor has been used, which is represented by the formula:

$$\text{Percentage increase} = 1.5\ (\phi - 35) + \frac{E}{500}.$$

This gives the percentage by which the total winter precipitation, November to April, inclusive, measured by the rain-gage can method, is to be increased to obtain the estimated true winter precipitation. ϕ is the latitude of the station and E is its elevation, feet above tide. For the author's station this formula gives a correction of 12 per cent which is the same as assuming

that three-fourths of the winter precipitation falls as snow, with a correction of 16 per cent for snowfall. So far as tried out the use of this correction has had the effect of making the winter precipitation consistent with the measured run-off of northern streams, with an allowance of 3 in. to 6 in. for winter water losses. Of course the correction should only be applied to stations where snow measurement is made by the gage-can method.

Subdivision of the available records for a given drainage area into summer and winter seasonal totals will sometimes show a marked deficiency in winter precipitation at certain stations without apparent reason, indicating cases where the correction is clearly required. If it is known that a rain gage has been located at a height above ground greater than 4 or 5 ft., or is in a very exposed position, a greater correction than that indicated by the formula may be required. If the height of the gage is known the correction may be made proportional to the square root of the height of the gage can above ground, as in the Abbe formula for wind effect. For example, if the calculated correction is 12 per cent for a gage 4 ft. above ground, as used in the experiments, then for a gage 9 ft. above ground the correction may be taken as $12\ \sqrt{9/4} = 18\%$.

Critical Examination of Records.

Before going further it is worth while to make a critical examination of the available records as to their homogeneity and relative accuracy. The fact that two rainfall records, even when taken at closely adjacent stations, show different long-term means affords no certain proof that either record is in error. Experience indicates that closely adjacent gages will often commonly differ in yearly precipitation amounts by say 5 per cent, the differences increasing for shorter time intervals. These differences may be and commonly are due to real differences resulting from climatic and topographic conditions. One station may lie to the leeward of a well-developed thunderstorm breeding ground and so may receive a larger amount of summer-time convective rainfall than the other (ref. 6). One station may be so situated as to receive more orographic precipitation than the other. This effect is usually most pronounced in the winter. Serious errors in rainfall records are more likely to result from poor exposure or lack of shielding of the gage from the wind than from any other cause, although a rainfall record may be defective owing to the gage not standing perpendicular or, though rarely, owing to a leaky gage being used for a long time without repair. Leaks in outer gage can not infrequently result from freezing of water in the gage in the winter when the inner tube is removed. Such a leak has no effect on the catch of snowfall unless the snow melts, and of course has no effect on the catch of rainfall when the inner tube is in use.

All the conditions above described tend to give too small a recorded rainfall. If the gage stands where it catches drip from trees, buildings or

electric wires, or if it is so close to the ground that snow drifts into it, it may give excessive results.

The usual height of exposure of a rain gage at voluntary stations is 4 to 6 ft. According to the Abbe formula (ref. 1), the percentage of catch of rainfall due to wind action where the gage stands above ground level is approximately:

$$\text{Per cent error} = 3.32\sqrt{H}.$$

Using a gage with the funnel one foot above ground as a standard, the error for a gage 4 ft. above ground, according to this formula, would be 6.64 per cent. This error will exist only in case the gage is wholly unprotected from the wind. Although this deficiency in catch of rain gages was experimentally proven in New England by Edmund B. Weston and has been repeatedly proven elsewhere, yet hitherto it has not been customary to correct rainfall records for such deficiency, partly because many rain gages are surrounded by shrubs and buildings, practically or at least partially equivalent to a Nipher shield. Nevertheless it appears certain that most rainfall records in the United States are slightly deficient as compared with a gage at ground level or a pit gage, such as used in England. Prof. A. J. Henry remarks (ref. 7):

"With regard to the elevation of the Weather Bureau gages high above ground on the roofs of large flat buildings . . . In general it does not seem possible to avoid the conclusion that the observed amount of precipitation falls short of the true amount by quantities varying from 5 to 10 per cent. of the annual rainfall."

In view of the fact that it is often difficult to determine precisely whether a correction is required or not, and since it would probably be impossible to have the correction for wind introduced as a general custom, no attempt has been made by the Committee on Water Losses to correct rainfall records for wind effect in the summer time unless there were specific data available indicating the nature and necessity of the correction required.

From the preceding it is evidently desirable in seeking the sources of differences between two rainfall records to compare them for winter and summer periods separately. Of course the difference between two records may be so great as to make it practically certain that one is in error, yet most persons will hesitate to reject a rainfall record altogether unless some reason is known for so doing other than the mere fact that it differs from a record at an adjacent station. If a rainfall record looks suspicious the first thing to seek is, of course, information regarding the exposure of the rain gage. Records sometimes lack homogeneity and show wider variations of rainfall than they should owing to changes in gage location. When an original gage location is found to be poor it is sometimes changed and no record made of the change. If a record is not homogeneous the fact will sometimes be revealed by plotting the record in parellel with those

for adjacent stations having homogeneous records or by comparing the ratios of the means for three, five or ten years' successive periods taken with relation to the long-term mean.

It is of course true that large variations in actual rainfall amount may occur within comparatively short distances. The rainfall at Honolulu is 25 in., that on the Koolau mountains five miles away is 100 in. (ref. 9). The case of Ben Nevis and Fort William, where there is an increase of 100 per cent in rainfall in four miles, has already been cited (ref. 10). Causal conditions for such variations are however obvious in the existing topography. Cases will arise where there are no adequate climatic or topographic reasons sufficient to explain existing differences in rainfall records at adjacent stations. There is no fixed rule by which it may be determined whether a given record which is apparently discordant should be rejected or not. There are, however, certain criteria which are indicators of the consistency and homogeneity of a given record. These include:

(*a*) The coefficient of variability.

(*b*) The range ratio.

Coefficient of Variability.

If in a series of N data, such as annual rainfall amounts, whose mean is M, the departures $\triangle$ of the individual values from the mean are determined, then the quantity

$$C_v = \frac{100}{M}\sqrt{\frac{\Sigma \triangle^2}{N}}$$

is called the coefficient of variability. The quantity under the radical is the standard deviation. The coefficient of variability is the ratio of the standard deviation to the mean, expressed as a percentage of the latter. If expressed merely as a ratio and not as a percentage the resulting factor may be called simply the "variability." The variability is a valuable index of the dispersion or scattering of a series of events about the mean.

Since, as already pointed out, the relative rainfall amounts at adjacent stations for the same time intervals usually agree closely, it would be expected that the variability for reliable rainfall records in a given region should be of the same order of value. This is found to be the case. Mr. Allen Hazen (ref. 8) has published a map deduced from the longer rainfall records, which is reproduced as Fig. 6, showing the variability of rainfall throughout the United States. East of the Mississippi the variability factor has values usually between the limits 0.14 and 0.18. The variability is largely a regional factor, depending on the structure of the climate rather than on the rainfall amount, although it varies with the latter indirectly, since the same causes which produce low rainfall usually produce high variability. In the Colorado River valley of Arizona and southern California, with a rainfall around 5 in., the variability reaches the maximum

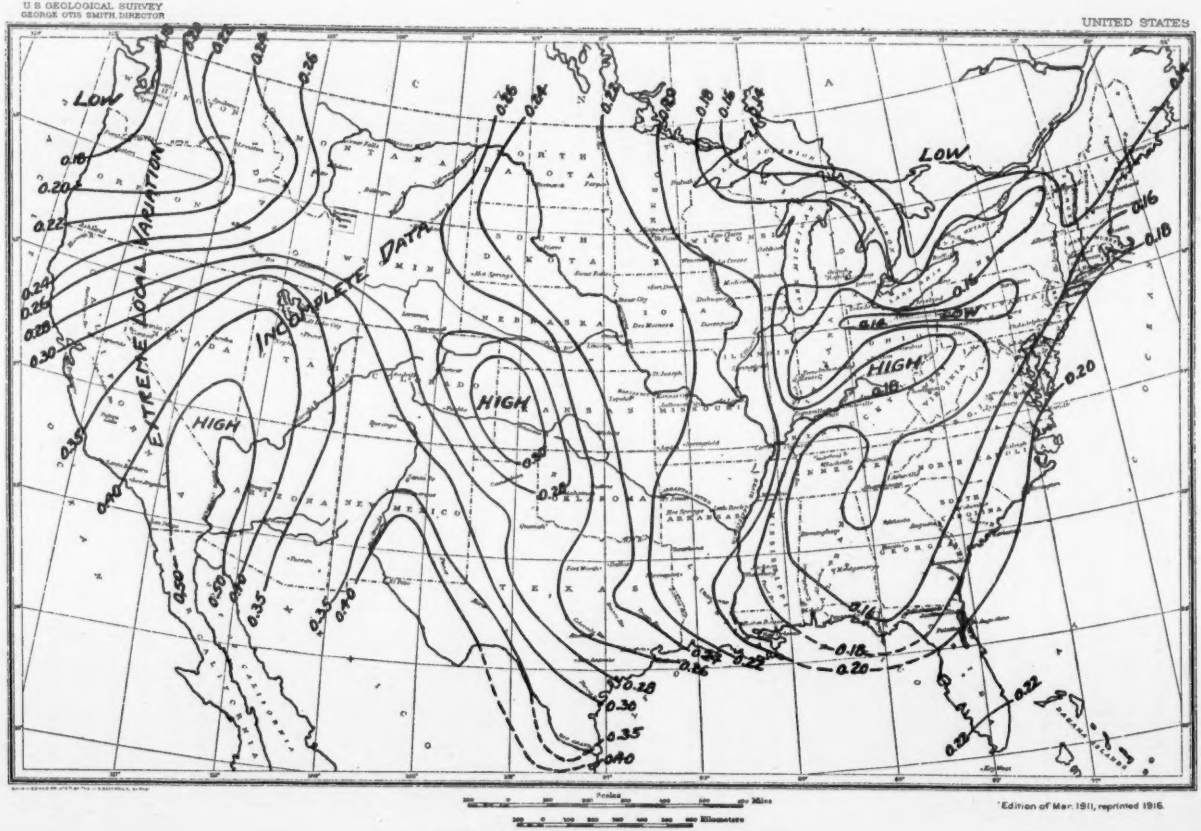

FIG. 6.— VARIABILTY OF PRECIPITATION.
(Hazen, *Engineering News*, January 6, 1916.)

value for the United States of 0.50. On the northwestern Pacific Coast in the State of Washington, with over 100 in. rainfall or more than 20 times as much as in the Colorado valley, the variability is 0.18. The lowest variability in the United States is 0.14 in northern New England and along the northern margin of the Great Lakes. There is some resemblance between a map of rainfall variability and one of thunderstorm frequency. In general, regions where a large portion of the summer precipitation is of convective origin are those of high variability. This would be expected, since thunderstorm rainfall is much more erratic in its occurrence than purely cyclonic or cyclonic-orographic rainfall. A rainfall record, like any other system of human measurement, cannot be held free from errors. The variability factor shows the resultant effect of variations in rainfall due to natural causes and also variations due to errors. If the errors are large the calculated variability will usually be abnormally high as compared with that prevailing generally in the same region. Since the true variability of rainfall in a given region is nearly constant, the variability may be used as a criterion to test the accuracy of a rainfall record. Unfortunately, it requires a comparatively long record to give the correct value of the rainfall variability at a given station. The shorter the record the higher is the apparent variability. If the record is accurate and homogeneous the variability should approach the normal for the region as the duration of the record increases. If there has been a change in exposure or other conditions it affects the homogeneity of a rainfall record and it is quite certain to be reflected in a high variability factor.

There are in New England several very long rainfall records. Data for eleven such records, each 57 years or more in length, are given in Table 5. The variability is below 0.16, with two exceptions, Albany and Boston, and each of these records is composite, i. e., it is made up by piecing together information of simultaneous records by different observers with different rain-gage exposures. These data indicate that in northeastern United States a rainfall record of thirty or more years' duration showing a variability factor exceeding say 0.20 demands critical examination.

Use of the Range Ratio.

If a series of annual rainfall records is plotted with the years arranged in order of magnitude, a curve drawn through the plotted points may be used as an integral frequency curve. Thus if there are 100 years of record, the 10th highest value will have been equaled or exceeded on an average once in 10 years, the 20th highest once in 5 years, and so on. Plate I gives examples of such curves, the annual rainfall amounts being plotted in terms of their ratios to the mean. The left-hand portion of the curve, corresponding to the shorter and better determined exceedance intervals, is smooth and uniform; the less frequent higher values to the right are more scattered, owing to the fact that some of these have occurred out of

TABLE 5.

Variation and Limitations of New England Rainfall.

Station.	Record.			Mean Rainfall In.	Variability Factor	Maximum Year.			Minimum Year.			Exceedance. Formula.		Subceedance. Formula.		Record Used.
	From	to	Yrs.			Year.	Ratio to Mean.	A Calc'd In.	Year.	Ratio to Mean.	B Calc'd In.	K	n	K	n	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Albany, N. Y.	1826	1913	88	37.98	.1627	1871	1.49	1.65	1913	0.69	0.68	.400	.32	1.120	.32	U. S. Wea. Bur., etc. (Composite Record).
Amherst, Mass.	1836	1913	78	44.15	.1268	1888	1.31	1.50	1908	0.70	0.60	.588	.26	.830	.26	Hatch Exp'm't Sta.
Boston, Mass.	1818	1913	96	45.46	.1766	1863	1.49	1.58	1822	0.61	0.55	.564	.33	.955	.20	Hale-Hall — City Main Drainage.
Burlington, Vt.	1838	1913	76	32.31	.1574	1869	1.43	1.50	1894	0.71	0.70	.502	.35	1.070	.32	U. S. Wea. Bur. P = 22.62 in 1914 = B.
Cambridge, Mass.	1841	1913	73	43.86	.1555	1859	1.36	1.36	1910	0.69	0.45	.874	.37	.506	.30	Harvard Coll. Observatory.
Cornish, Me.	1857	1913	57	46.19	.1377	1902	1.37	1.60	1880	0.72	0.60	.435	.35	.973	.18	U. S. Wea. Bur.
Gardiner, Me.	1837	1913	77	43.30	.1300	1887	1.26	1.27	1838	0.70	0.69	.765	.43	.669	.36	U. S. Wea. Bur.
Lowell, Mass.	1826	1913	88	41.92	.1565	1888	1.42	1.45	1846	0.67	0.60	.662	.30	.794	.26	Merrimac Mfg. Co.
New Bedford, Mass.	1814	1913	100	46.47	.1433	1829	1.41	1.45	1846	0.74	0.74	.387	.46	1.196	.25	Rodman and Hathaway. P = 32.90 in 1918. B = 34.38.
Providence, R. I.	1832	1913	82	45.01	.1460	1898	1.41	1.50	1846	0.68	0.60	.610	.26	.847	.26	Brown Univ. — Hope Reservoir.
Waltham, Mass.	1825	1913	89	42.74	.1499	1850	1.45	1.66	1846	0.66	0.58	.419	.28	1.049	.20	Boston Mfg. Co.

their turn. For example, a 100-year record may contain a year with rainfall so great that it will not be exceeded as often as once in 200 years, or it may not contain as many as five 20-year exceedance intervals.

The Gaussian law of error and various modifications thereof have been used to represent frequency curves of rainfall. Such formulæ seem to the author defective for the reason that they are in a form which indicates the possibility of an infinite value of the event if the record or exceedance interval is only made long enough. To use a homely, but I think very appropriate, illustration, there is just as much probability of a rainfall of 1 000 in. in a year at Boston, — the climate remaining the same, — as there is that an ordinary domestic hen will lay an egg a yard in diameter. Both are impossible and for very much the same reasons. The hen is not large enough and in the case of rainfall nature's capabilities are inadequate, — the available moisture supply is not large enough. Assuming the truth of the proposition, which probably every one will accept, that there is some, though perhaps unknown, limit to the amount of rainfall, either maximum or minimum, which nature can produce in one year at a given place as long as general climatic conditions remain as they are, then it is evident that as the exceedance interval increases, the exceedance curve, Plate I, should approach a horizontal or asymptotic line representing this maximum value.

If an equation can be found which accurately represents the left-hand portion of such a curve or the portion corresponding to the greater frequencies or shorter exceedance intervals, this equation being of such form that it approaches a horizontal or asymptotic line as the exceedance interval increases, then the maximum or limiting value of the ratio of the annual rainfall for any one year to the mean can be determined. Plotted in this form, however, the data do not afford a basis for determining the minimum ratio of the annual rainfall to the mean. In order to determine the latter the data are again to be arranged in order of magnitude, but this time beginning with the lowest annual rainfall and the average time intervals at which rainfall amounts having less than a given ratio to the mean computed. The average interval at which an event may be expected to occur having a value less than some assigned value is designated for convenience the subceedance interval.

In such a curve the annual rainfall ratios also approach an asymptotic line as the subceedance interval increases. A pair of such curves affords a basis of determining both the maximum and the minimum annual rainfall which can occur at a given station. An expression, the graph of which fits the better determined portion of the rainfall exceedance curve and meets the requirement of approaching a horizontal asymptote as a limit, as the exceedance interval increases, is,

$$R = B + (A - B)\left[1 - \epsilon^{-k(t-1)^n}\right]$$

and for subceedance curves,

$$R = B + (A - B)\,\epsilon^{-k(t-1)^n}$$

where R is the ratio of the rainfall having an exceedance or subceedance interval to the mean rainfall. A, B, k, and n are constants to be evaluated from the better determined portion of the curve. The line CD, Plate I, was plotted from the formula deduced for Boston. It will be noted that the calculated curve and the plotted points are in almost perfect agreement for all but a few of the higher values of R, representing, as already explained, rainfall amounts which have occurred out of their turn. The method of calculating the constants in this formula is somewhat involved and need not be detailed here. The point to which attention is especially directed is that the procedure described affords a method of determining from the more frequent and most certain values the frequency equation for a given rainfall record. This frequency equation contains as necessary constants values A and B, representing approximately at least the greatest and smallest annual amounts of rainfall which can ever occur at the given station, whether the rainfall record used happens to contain actual values closely approaching these limiting values or not. The constants in the frequency equation above given have been determined for eleven long New England rainfall records and are given in columns (9) and (12) of Table 5.

In finding the constants, a value of A or B is assumed and values of the other constants calculated. If the resulting values of R are in good agreement with the left-hand portion of the curve, A must be about correct. However, a variation of 5 or 10 per cent in the value of A from the true value is often about the least which will produce a noticeable departure of the calculated from the observed curve. This represents the limit of accuracy to which the maximum and minimum possible values of R can be determined. It is therefore always possible that an annual maximum rainfall 5 to 10 per cent greater or 5 to 10 per cent less than that indicated by the formula may occur.

It will be interesting to see how the maximum and minimum observed yearly rainfall amounts in these long records compare with the maximum calculated maximum and minimum possible amounts. At Boston, for example, the highest observed rainfall in 88 years is 1.49 times the mean; the calculated maximum is 1.58 times the mean, but since this may be in error at least 0.05, a rainfall of 1.63 times the mean appears possible. The least rainfall that has occurred is 0.61 of the mean; the value of the constant B is 0.55 of the mean, and the least possible amount may be taken as 0.50 of the mean. In several instances the highest and lowest values ever observed in these long records are considerably short of the limiting values. In other cases values closely approximating the limiting values have been observed.

The application of these results to the matter of testing the validity of any rainfall record may now be considered. For this purpose it is best to use the maximum range ratio or ratio of the difference between the limiting maximum and minimum annual rainfall amounts to the mean. The observed ranges and the calculated and limiting ranges for the eleven

stations are given in Table 6. In no case has the actual range ratio been as great as unity. The calculated maximum range ratio obtained by taking the difference between the constants A and B is usually near unity and sometimes slightly exceeds it. While there are reasons for believing that the differences in the constants and resulting range ratios for the several stations are due to actual differences in conditions governing rainfall, yet since none of the calculated range ratios materially exceeds unity it is safe to assume that if anywhere in New England a rainfall record is found in which the range ratio much exceeds unity, the record is open to

TABLE 6.

RANGE OF ANNUAL RAINFALL IN NEW ENGLAND.

PER CENT OF ANNUAL MEAN.

Station.	Years of Record.	Observed Range.	Calculated Range A—B.
(1)	(2)	(3)	(4)
Albany, N. Y.	88	.80	.97
Amherst, Mass.	78	.61	.90
Boston, Mass.	96	.88	1.03
Burlington, Vt.	76	.71	.80
Cambridge, Mass.	73	.67	.71
Cornish, Me.	57	.65	1.00
Gardiner, Me.	77	.56	.58
Lowell, Mass.	88	.75	.85
New Bedford, Mass.	100	.67	.71
Providence, R. I.	82	.73	.90
Waltham, Mass.	89	.79	1.02
AVERAGE		.70	.86

suspicion. This criterion is better than that of variability. It is simple, direct and applies to a short as well as to a long record. Unfortunately, limiting values of the range ratios are not available outside of New England. For the eleven New England records given the average calculated range ratio is 0.86 and the average observed range ratio is 0.70. Applying the ratio of these quantities to the maximum observed range ratios derived from long records in other regions, an approximation of the limiting ratios elsewhere may perhaps be obtained. The range ratio usually increases as the duration of the record increases, approaching its limiting value only for a very long record. The range ratio is related to the mean rainfall and also to the rainfall variability.

It has no doubt occurred to you that if so much preliminary work must be done, the task of determining the average precipitation over a drainage basin would be endless. Fortunately the critical analyses last described need be made only in occasional instances. The criteria given will not

always serve to confirm or condemn a record that appears suspicious, but they are the best that are available.

One more correction may be required, namely, for elevation. Consideration of this will be deferred until after the discussion of methods of calculating areal mean rainfall.

Calculation of Areal Mean Rainfall.

Arbitrary Weighting of Stations. Having the records compiled and corrected (except for elevation), the method of combining them to obtain the best value of the areal mean is next to be considered. If the area is large and rainfall records within the area numerous and evenly distributed, a direct average may be used. For medium and small-sized areas the use of some records lying outside the watershed boundary is often desirable and sometimes unavoidable. Furthermore, the stations will not, as a rule, be distributed geographically in such a manner as to justify giving them equal weights. Sometimes the different records are assigned arbitrary weights fixed by judgment. For example, an outside station might be given a weight one-half as great as a station within the watershed line. It is often difficult to determine proper weights by unaided judgment and different computers will usually not arrive at the same values (ref. 11).

Thiessen's Method. The fundamental principle of the Thiessen method (ref. 12 and 13) is that a rainfall record at any given station shall be used to represent the precipitation on that portion and only that portion of the drainage basin to which it lies nearer than any other station.

In the application of the method a small scale outline of the watershed boundary should be drawn and the different rainfall stations within and adjacent to the area located on the same map. Fig. 8 illustrates such an outline map for a drainage basin having one station within the area, one on the margin and several outside. Lines connecting the different stations are drawn, as 1-2, 2-3, etc., and perpendiculars are erected at the median points on these lines, as AB, BC. EF and EK. Station 1 lies nearer than any other station to all points within the area enclosed by the dashed line GABEF and the portion of the watershed boundary FG. Station 2 lies nearest to the portion of the area GBHG, and station 3 lies nearest to the portion HBEKLMN. Points on the line GB are equidistant from stations 1 and 2. Stations 6, 7, and 8 are not applicable to any portion of the area and may be eliminated. This is evident, for example, in the case of station 6, since the perpendicular OP at the median point of the line 6-3 lies wholly outside the area. Points on this line are equidistant from stations 3 and 6. Hence no part of the area is as near to station 6 as to station 3. The subareas representing portions of the drainage basin nearest to each individual station may be determined by planimeter and the corresponding percentages of the total area computed. These percentages of the total area represent the weights to be assigned to the corresponding stations. The

graphical construction is easily carried out for any form of area and combination of stations after a little practice. One advantage of the method is that the determination of weights is a purely geometrical process and different persons will obtain the same results. Since perpendiculars erected

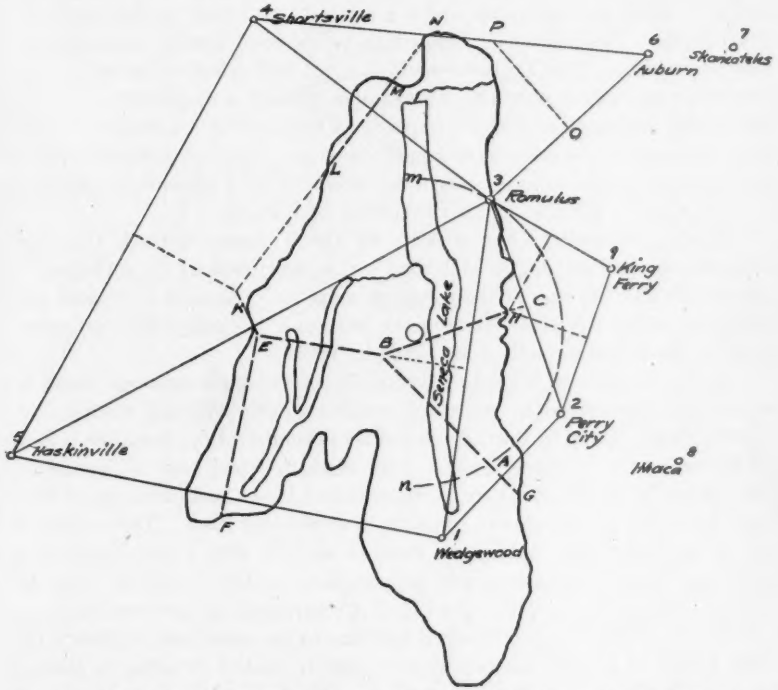

FIG. 8.— GRAPHICAL METHOD OF SELECTING RAINFALL STATIONS APPLICABLE TO A DRAINAGE AREA, AND OF DETERMINING THEIR RELATIVE WEIGHTS.

	Sq. Miles.	Per cent.
Wedgewood	341	42.6
Perry City	76	9.5
Romulus	344	43.0
Shortsville	11	1.4
Haskinville	28	3.5
	800	100.0

at the median points of the three sides of a triangle intersect in a common point it is evident that the perpendiculars to the lines connecting any group of three stations will so intersect. This is illustrated by the point B in the case of the triangle 1-2-3, Fig. 8. The object of the geometrical construction is to determine the subarea which lies nearest to each individual station. The results can readily be checked by setting the point of a

compass at any location, such as s, which is in doubt, and then drawing an arc through what appears to be the nearest station, — in this case, Romulus. The arc MN shows that Romulus is the nearest station and therefore the point s is properly located within the Romulus subarea. It will subsequently be shown that the Thiessen method is best adapted for use in the case of a moderate-sized area where a limited number of rainfall stations is available. That, however, happens to be the most typical and common case in practice. The Thiessen method is not well adapted for use in the case of a very small area where, for example, there is a single station at one side of the drainage basin which happens to be nearer to all portions of the area than any other station, although there may be good stations a little more remote on the opposite side of the area. In such an instance stations on both sides of the area should in general be utilized.

Having determined the weights by the Thiessen method, the precipitation at each station for each time unit is multiplied by the corresponding weight and the sum of the resulting weighted quantities is used as the precipitation on the area for the given time unit. Examples of computation by the Thiessen method are given in Table 8.

Center of Gravity Method. The mean rainfall on a drainage basin is sometimes determined by assigning weights to the different records inversely proportional to the distances of the several stations from the center of figure of the drainage basin. This method, which was suggested to the author by W. W. Peabody, is well adapted to use in a case such as that last described, but is open to question in some other cases. There appears to be no reason for applying to stations that lie well within limits of a drainage basin weights inversely proportional to their distances from the center of figure of the area. In Fig. 9 a comparison of the application of the center of gravity and Thiessen methods to the same area is illustrated. The center of gravity method gives weight to station A equal to that of station B, although station A is outside the drainage basin. It gives a weight to station A greater than to station D, although the portion of the area lying nearest to station D is larger than that lying nearest to station A. The Thiessen method automatically eliminates outside stations unless they lie nearest to some portion of the drainage area. By the center of gravity method there is no restriction as to the stations to be included, the limiting distances of stations used being determined by judgment. In general if two stations are nearly in the same line, one more remote than the other, only the nearer station would be utilized.

Topographical conditions may be such that the use of the Thiessen method is not desirable. Such a condition exists in the case of some small basins feeding the lower Hudson River from the west, and illustrated schematically by Fig. 10. Here part of the drainage basin lies east of and below the Helderberg escarpment AB. To this portion station C may be applied, and although this station lies nearest of any to the part of the area west of AB it is not applicable above the escarpment AB. The rain-

fall west of this escarpment is much higher than at C, but quite uniform. The records for stations D and E may be applied to this part of the area with weights inversely proportional to their distances from the center of Fig. F of the upper area.

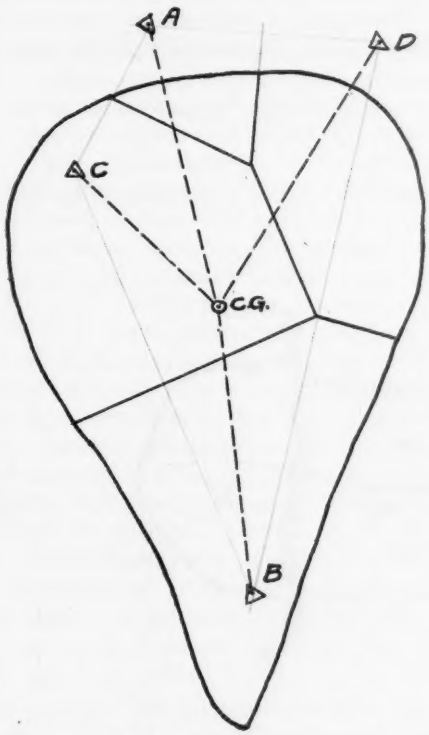

FIG. 9. — RELATIVE WEIGHTS OF RAINFALL STATIONS.

Station.	Thiessen Method.	Gen. Grav. Method.
A	4.09 per cent	23.16 per cent
B	37.47 „ „	23.16 „ „
C	38.87 „ „	32.63 „ „
D	19.57 „ „	21.05 „ „
	100.00	100.00

Method of Isohyetal Lines. The rainfall contour map affords an excellent mode of graphical presentation of rainfall distribution over a drainage area. Such maps are sometimes used as a basis of determining the areal mean rainfall by planimeter measurement of the areas between successive isohyetal lines.

Let a_1, a_2, a_3 etc., be the areas between isohyets P_o, P_1, and between P_1, P_2, etc.

Let

$$P'_1 = \frac{P_o + P_1}{2},\ P'_2 = \frac{P_1 + P_2}{2},\ \text{etc.},$$

then the mean precipitation P_m on the area is

$$P_m = \frac{1}{A} \Sigma a P'$$

where $A = a_1 + a_2 + a_3$, etc., = the total area.

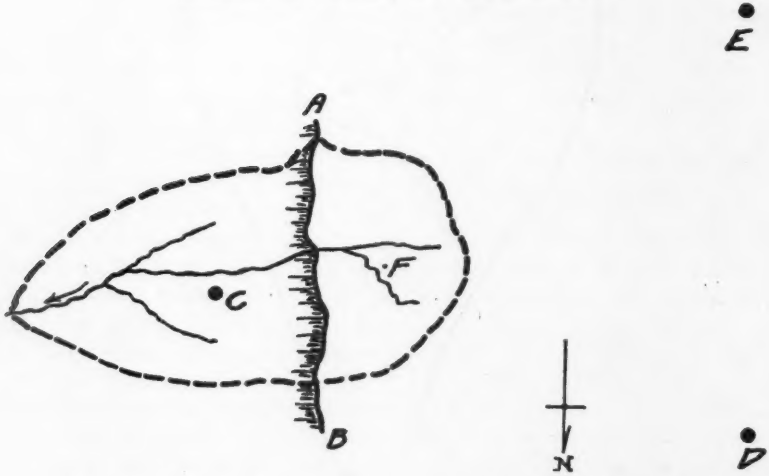

FIG. 10.— EFFECT OF MOUNTAIN ESCARPMENT ON RAINFALL DISTRIBUTION.

The determination of the areal mean rainfall by this method for a long series of individual months is very laborious, since it involves drawing a separate rainfall contour map for each month. The preparation of such maps is only practicable where there is a sufficient number of rainfall stations within and closely adjacent to the drainage basin.

Dr. Julius Hann (ref. 7) states:

"My opinion is that there is no well-established basis for drawing isohyetal charts for individual months — having regard to altitude, wind direction, or the distribution of forests — that shall represent anything more than a subjective work of the imagination. In so far as the distribution of rain thus presented departs from the quantities actually measured it is arbitrary, and every draftsman will produce a different picture. Such charts may even be hurtful and productive of error, if it is forgotten that

in many places they are purely hypothetical. When one looks at something that is well drawn, he generally assumes it to be correct. Even the expert cannot, of course, without some reserve, divest himself of the impression made by such a chart."

While the contour method may be theoretically the most accurate of any yet devised, there is a question whether its accuracy may not be somewhat fictitious, especially when the number of rainfall stations is meagre. In such a case two computers may differ in the location of contours. Furthermore, in drawing isohyetal maps the rainfall contours are usually assumed to occupy the same positions which they would occupy if they were really ground surface contours based on elevations corresponding to the measured rainfall at the different stations. If rainfall varies with the topography the contours so drawn may not be in their true locations.

Precipitation on Small Areas at High Elevations. Cases requiring the determination of rainfall on a small area and at a high elevation where there is no rainfall station within the area are perhaps more apt to arise in connection with water-works practice than elsewhere, since such basins frequently form the sources of gravity water supplies. Problems will arise that tax the ingenuity and they must be solved, if at all, by methods developed to meet the particular conditions and data available.

An ingenious method of determining the mean rainfall on small areas draining high mountain slopes of the Sierras has been suggested by Charles H. Lee (ref. 14). The rainfall data are meagre and insufficient for independent determination of the areal mean precipitation on the small individual basins. The relation of precipitation to elevation is, however, quite uniform and has been determined. The areas have characteristic forms consisting of triangles with their bases along the crest of the mountain range, alternating with lower-lying triangular basins with their vertices upward. For uniform variation of precipitation with elevation in such a case, the mean precipitation on the drainage basin is the same as that at the center of figure of the area. This for a triangular area is one-third the distance from the base to the apex. Thus for a basin with apex upward, the mean precipitation is the same as that at one-third the distance up the slope, while for a basin with apex downward the mean is the same as for a precipitation two-thirds the distance up the slope from the apex. It is interesting to note that if two such equal adjacent basins extend between equal limits of elevation they will have considerably different areal mean rainfalls. For example, if the rainfall increases from 10 in. at the foot of the slope to 25 in. at the headwater divide, the mean rainfall on a basin with apex upward will be 15 in., while on an adjacent basin of equal width with apex downward it will be 20 in. Two such basins would differ materially in run-off, although equal in size, identical in shape and slope, and although rain gages at the same elevations in both basins would give the same results.

It is evident that no single method of determining areal rainfall means is adapted to all cases. The method to be used in each instance depends upon:

(1) Size, shape and topography of the drainage basin.
(2) Number and positions of the rain gages in relation to the area.
(3) Accuracy obtainable and accuracy desired in the result.
(4) Labor involved in the reduction of the data.

As elsewhere in hydrologic work, the principle of making use of all the available information should in general be followed. Fragmentary records are better than none, but they may be disregarded when there are closely adjacent continuous records or when the number of records available is more than sufficient to secure the desired degree of accuracy.

Correction of Rainfall for Elevation.

If a drainage basin is oriented in the direction from which the prevailing rain-bearing winds are derived, the rainfall usually increases with elevation This is not always true, since elevation usually increases proceeding inland and since a portion of the moisture providing rainfall is usually derived from the ocean it may happen that the decrease in precipitation proceeding inland as a result of increased dryness of the air may equal or overbalance the increase due to higher elevation. This occurs, for example, in the Mississippi Valley, where the increase of elevation is so gradual that the rainfall decreases proceeding inland or upstream from the Gulf of Mexico.

It often happens that the majority of the rainfall stations available for use in determining the average precipitation over a given drainage basin are located at the lower elevations. Rainfall stations are more common in stream valleys than at higher elevations, since most cities and towns are in the valleys. If rainfall increases with elevation and the mean elevation of the rainfall stations is not the same as the mean elevation of the drainage basin, a correction of the rainfall records for elevation is obviously desirable. The correction of rainfall for elevation is most necessary in case of rainfall stations lying outside the drainage basin. It may, however, be necessary also for stations within the watershed boundary. In applying the Thiessen method each rainfall station is given a weight proportional to the part of the drainage basin to which it lies nearest. Frequently the elevation of the rainfall station is not the same as the mean elevation of the subarea to which it applies and an elevation correction is needed.

It is convenient to consider two classes of cases:

(1) Those where there is a definite and regular relation of precipitation to ground elevation.

(2) Those where the rainfall is related to what may properly be termed the dominant elevation rather than to the actual ground elevation at the rainfall stations.

The first condition, namely, that of a more or less definite relation of precipitation to ground elevation, is most likely to occur where a drainage

basin is oriented so that the prevailing slopes are toward the direction from which the principal rain-bearing winds are derived. In this case a consistent rain-elevation correction curve can usually be obtained by plotting the mean precipitation for simultaneous intervals at different stations as ordinates in terms of the elevations of the stations as abscissas. It is sometimes desirable to prepare separate curves for the summer and winter seasons, since the proportion of orographic precipitation and hence the increase of precipitation with elevation is generally greater in winter than in summer. Having given such correction curves, the precipitation at each station can if desired be corrected to mean elevation of the basin before applying the data to the calculation of the areal mean precipitation. If the Thiessen method is used the precipitation for each station can be corrected to the mean elevation of the subarea to which the station applies. Correction of the records for the individual stations involves, however, unnecessary labor, since it is usually possible to correct the areal mean precipitation for a given month or season directly without correcting the individual station records. If the several rainfall records are given equal weights the correction factor is obtained by finding from the rainfall elevation curve the ratio of the average precipitation at the mean elevation of the rainfall stations to the average precipitation at the mean elevation of the drainage basin. The areal mean is multiplied by this ratio as a correction factor.

If the rainfall stations are given different weights, as for example, in the Thiessen or center of gravity methods, a blanket correction can still be applied, but in this case the weighted mean elevation of the stations is to be used instead of their average elevation. This is obtained by multiplying the elevation of each rainfall station by its weight and dividing the sum of the products by the sum of the weights, if other than unity. The ratio of the average precipitation corresponding to the weighted mean elevation of the rainfall stations to that corresponding to the mean elevation of the drainage basin is then determined and applied as a correction factor, as before.

Use of Dominant Elevations. If a stream valley is so oriented that the prevailing rain-bearing winds blow across rather than with the streams, then records for rain gages located in the valleys may show no consistent relation to their actual elevations, although there may be a general increase in precipitation with elevation for the basin as a whole. In such cases a low-lying valley station may show about the same precipitation as a much higher station on the margin of the valley in line with the same air current. Here the amount of rainfall at either station is controlled not by the actual ground elevation but by the highest or dominant elevation over which the rain-bearing wind must pass before it reaches the stations.

The upper Rondout or Lackawack drainage basin in the southern Catskill region of New York affords an illustration of this condition. The locations of rainfall stations are shown on Fig. 11. Using Lackawack as

a base station, the ratio of the annual precipitation for each year and station to that for the same year at Lackawack was computed. The means

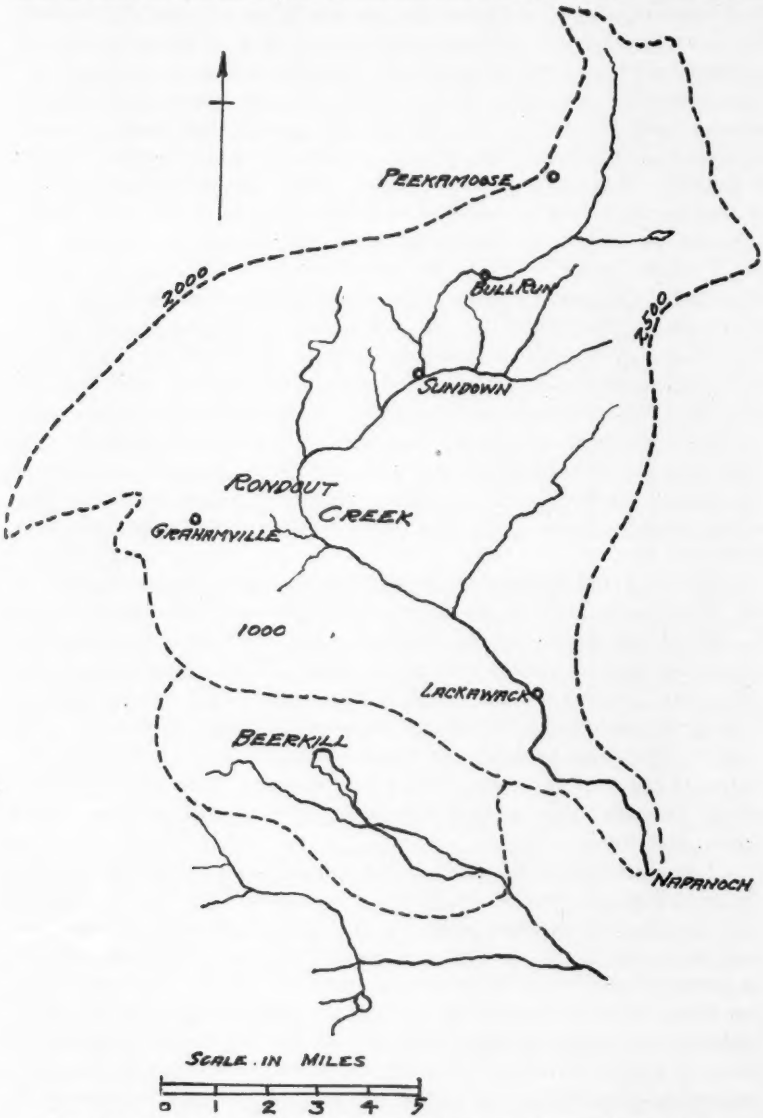

FIG. 11.— LOCATION OF RAINFALL STATIONS, RONDOUT CREEK BASIN, N. Y.

of these ratios for periods when rain gages were at different locations and elevations are shown in column (4) of Table 7. It will be noted that there

is no consistent relation between the ratio to Lackawack and the elevation of a given gage. In some instances the ratio is higher and in some instances lower for the higher of two gages. This suggests that the dominant topography rather than the precise elevation of the gage in the Rondout Valley is the determining factor.

From a careful study of the U. S. Geological Survey topographic map the dominant elevation corresponding to each gage was determined. This

TABLE 7.

RELATIVE RAINFALL AT STATIONS ADJACENT TO BEERKILL DRAINAGE BASIN.

Station.	Period.	Actual Elevation.	Yearly Ratio to Lackawack.	Dominant Elev.	Ratio to Lackawack.	
					Summer.	Winter.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Napanoch	1907–1909	600	1.00	600	1.00	1.00
Lackawack	1909–1921	620	1.00			
Rosendale	1914–1917	100	.92	100	.89	.96
High Falls	1907–1908	220	0.99	300	.95	.98
	1908–1916	320	0.95			
Grahamville	1907–1913	900	1.053	1000	1.08	1.13
	1914–1921	810	1.07			
Tabasco	1907–1908	990	1.07	990	1.07	1.08
Sundown	1907–1915	960	1.015	1500	1.04	1.08
	1916–1917	1140	1.09			
	1918–1921	1550	1.15			
		1620	1.06			
Peekamoose	1908–1911	2300 summ. 1680 win.	1.40	2500	1.28	1.34
	1912–1921	1440	1.26			
Claryville	1907–1910	1750	1.115	2000	1.09	1.12
	1911–1917	1590	1.09			

was taken as the least elevation to which a moisture-laden air current from the south or southeast must rise over intervening topographic features before it can reach a point over the gage. The dominant elevation may be higher but cannot be lower than the gage elevation. For most of the gages, with the exception of those at Sundown and Peekamoose, the dominant and actual elevations are about the same. Plotting the ratios of precipitation to that at Lackawack in terms of dominant rather than actual elevation, a fairly consistent and nearly lineal increase of rainfall with ele-

vation is obtained, Fig. 12. The ratios to the precipitation at Lackawack for summer and winter seasons can be expressed in the form:

$$\text{Summer} \ldots\ldots \frac{P}{P_1} = 0.91 + 0.15 \frac{H}{1\,000}$$

$$\text{Winter} \ldots\ldots \frac{P}{P_1} = 0.925 + 0.168 \frac{H}{1\,000}$$

P_1 being the precipitation at Lackawack, elevation 600, and H the dominant elevation controlling the air currents blowing over a given portion of the region.

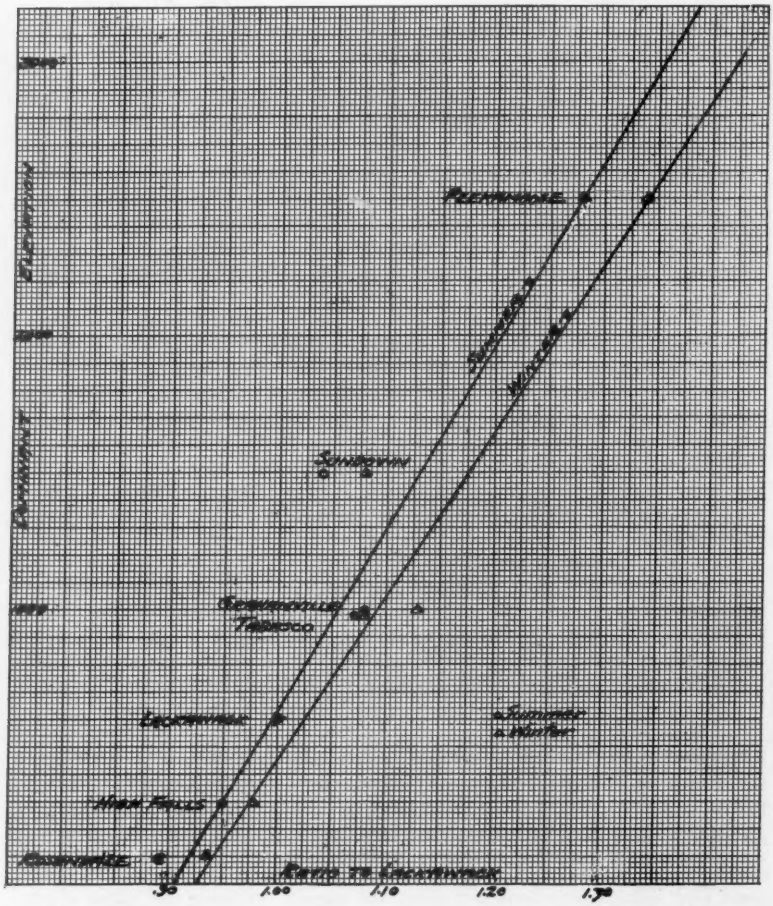

FIG. 12.— RATIO OF PRECIPITATION AT RONDOUT BASIN STATIONS TO PRECIPITATION AT LAKAWACK.

TABLE 8.

Weighted Monthly Precipitation.

Saranac Drainage Basin, 1913.

(A) Station.	Wt.	Jan.		Feb.		Mar.		April		May		June		July		Aug.		Sept.		Oct.		Nov.		Dec.	
		P	P′	P	P′	P	P′	P	P′	P	P′	P	P′	P	P′	P	P′	P	P′	P	P′	P	P′	P	P′
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)*	(26)
(1) Faust	.115	5.83	.660	1.55	.178	4.53	.521	3.34	.384	3.48	.400	3.48	.400	4.89	.560	2.22	.255	3.18	.366	7.94	.913	3.69	.424	2.46	.283
(2) Gabriels	.534	5.45	2.910	1.69	.902	7.34	3.920	2.90	1.550	3.80	2.030	2.38	1.270	4.06	2.170	2.52	1.340	2.18	1.160	4.36	2.330	2.56	1.370	2.40	1.280
(3) Chazy	.017	1.50	.026	0.77	.013	4.13	.070	0.84	.014	2.36	.040	1.21	.020	1.78	.030	1.07	.018	2.31	.039	2.99	.851	0.22	.004	2.28	.039
(4) Harkness	.021	2.06	.043	0.78	.016	3.81	.080	1.82	.038	2.09	.044	1.45	.030	3.78	.079	1.15	.024	2.92	.061	3.94	.083	1.30	.027	0.99	.021
(5) Dannemora	.285	3.68	1.050	0.79	.225	3.09	.881	2.62	.747	2.24	.639	2.01	.573	2.34	.667	1.76	.502	2.83	.806	4.98	1.420	2.36	.673	1.87	.523
(6) Keene Valley	.028	5.25	.147	1.26	.035	5.54	.155	2.56	.072	2.62	.073	1.43	.040	3.43	.096	1.06	.030	2.76	.077	4.94	.138	2.64	.074	2.51	.070
(7) Total P′			4.836		1.369		5.627		2.805		3.226		2.333		3.602		2.169		2.509		4.935		2.572		2.216
(8) Distributed P′			5.00		1.55		6.73		2.65		3.48		2.18		3.72		2.31		2.000		4.00		2.35		2.20

(B) Ausable Drainage Basin, 1913.

(1) Harkness	.225	2.06	.464	0.78	.176	3.81	.857	1.82	.410	2.09	.470	1.45	.316	3.78	.850	1.15	.259	2.92	.657	3.94	.886	1.30	.292	0.99	.223
(2) Keene Valley	.439	5.25	2.300	1.26	.553	5.54	2.430	2.56	1.120	2.62	1.150	1.43	.628	3.43	1.500	1.06	.465	2.76	1.210	4.94	2.170	2.64	1.160	2.51	1.100
(3) Lake Placid	.336	7.36	2.470	2.27	.763	7.90	2.650	4.46	1.500	1.85	.622	2.22	.746	3.75	1.260	1.57	.528	2.69	.904	4.88	1.640	4.08	1.370	3.98	1.340
(4) Total P′			5.234		1.492		5.937		3.030		2.242		1.690		3.610		1.252		2.771		4.696		2.822		2.663
(5) Distributed P′			5.46		1.31		5.77		2.66		2.73		1.49		3.57		1.10		2.87		5.14		2.75		2.61

* Mean of Gabriels and Keene Valley.

A rainfall-elevation curve based on dominant elevations is used in the same way as the ordinary relation curve, except that the correction factor is determined from the dominant instead of the actual elevations, both of the rainfall stations and of the areas or subareas.

The Index Station Method.

It is much less laborious to calculate areal mean precipitation for winter and summer seasons only than for the individual months of the year. This is especially true where correction for snow catch is necessary. When it is desired to determine the precipitation on an area month by month, a convenient method is to first calculate the precipitation for the individual

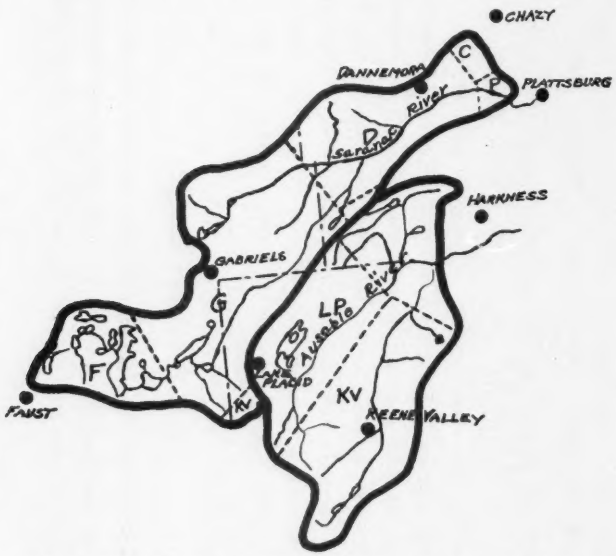

Fig. 13.— Drainage Basins and Rainfall Stations Ausable and Saranac Rivers.

winter and summer six months' periods and then distribute these results among the separate months by the Index station method. This method depends on the Kämtz principle already cited, and consists in finding the ratio of the precipitation in each month to the total for the season at one, two or three typical stations and multiplying the seasonal total by the mean of these ratios for each month and using the results as the monthly areal means. This will not give a precise determination of the precipitation for each month, but since the seasonal total is correct, the errors in monthly amount will compensate in the course of the season. The method is limited in its applicability. It is not desirable or economical to use it if the number of stations is very small, nor is it desirable to use it if the area is large and the number of stations so great that a direct average of the rainfall at the

different stations can be utilized. In such a case it is nearly as easy and more accurate to take a straight average of all the stations, month by month.

As an illustration of the application of the Index station method, Table 8 is given, showing the precipitation on the Saranac and Ausable drainage basins as calculated by the Thiessen method for the individual months of the year 1913. The locations of rainfall stations and the subdivisions of the area used with the Thiessen method are shown in Fig. 13. In Table 8, Section A, for the Saranac basin, line 7 shows the monthly precipitation by the Thiessen method and line 8 the monthly precipitation obtained from the annual total by the Index station method, using Gabriels as an index station. In Section B, Table 8, for the Ausable basin, the areal means by the Thiessen method are given in line 4 and the monthly means obtained from the annual total by the Index station method, using Keene

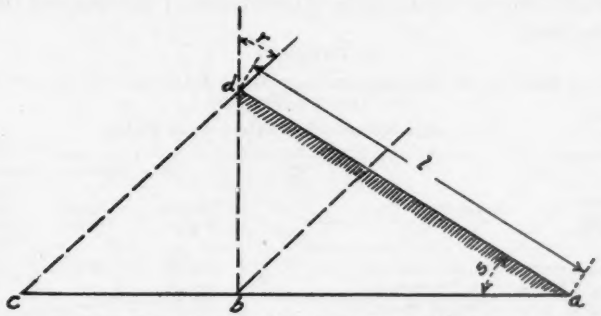

FIG. 14.— INCREASED CATCH OF PRECIPITATION ON MOUNTAIN SLOPES.

Valley as an index station, are given in line 5. For the Ausable the agreement between the two methods is invariably good. In the case of the Saranac basin, which is larger, better results would no doubt have been obtained by the use of two index stations. The saving in labor effected by the use of the Index station method is indicated by the fact that for the determination of the monthly precipitation on the Saranac basin in a single year by Thiessen's method 156 operations are required, whereas for the determination of the monthly precipitation by the Index station method only 36 operations are required.

ROOF EFFECT.

A condition may exist where the measured precipitation on a drainage basin, even though accurately determined, does not truly represent the amount of rainfall. Suppose, for example, a drainage basin is steeply inclined to the windward, as shown in Fig. 14. In the presence of wind, precipitation, particularly snow, may fall at a considerable angle to the vertical. Suppose this angle is $r = bdc$, Fig. 14. Then the precipitation actually caught on the inclined area *ad* is to the amount which would be

caught on the horizontally projected area ab as ac is to ab, but $ab = l \cos s$; $bc = l \sin s \tan r = l \frac{\sin s \sin r}{\cos r}$.

$$ac = ab + bc = \frac{l}{\cos r}(\cos r \cos s + \sin r \sin s)$$

$$= \frac{l}{\cos r} \cos (r\text{-}s) \text{ and}$$

$$\frac{\text{Catch}}{\text{Measured Precipitation}} = \frac{\cos (r - s)}{\cos s \cos r}$$

In case the area does not face the wind directly the angle s can be determined by the formula:

$$\tan s = \tan a \cos b$$

where a is the true or maximum slope angle of the area and b is the horizontal angle between the direction of orientation of the area and the direction of the wind.

TABLE 9.

SUMMARY OF RESULTS OF DETERMINATION OF MEAN RAINFALL ON SCITUATE BASIN BY VARIOUS METHODS.

Note.—ALL RECORDS EXTENDED TO 29 YEARS.

Stations Used.	MEAN RAINFALL—INCHES.		
	Thiessen Method.	Contour Method.	Weighted Inversely as to Dist. from C. of G.
All having 2 years or longer records	45.91	45.77	46.00
All having 8 years or longer records	46.26	—	45.69
All having 27 years or longer records	46.33	—	45.76

Mean of all = 45.96 inches.

Snow often falls at large angles to the vertical, sometimes 70 to 80 degrees. For a steep area facing the wind the snow precipitation may therefore easily be 1.5 to 2.0 times the measured precipitation. An instance where this effect apparently exists occurs along the west slope of the Continental Divide in Colorado. There a number of records of the yield of small streams accurately gaged show an average annual run-off of 22.5 in. as compared with a precipitation on their drainage basins, corrected for elevation, of about 30 in.* Some of the rainfall stations are very close to the divide, others are within and adjacent to the stream basins. Most of the precipitation falls as snow, accompanied by wind blowing toward the mountain slopes. The conditions are known to be favorable for the occurrence of a roof effect such as has been described, and this is apparently the only tenable hypothesis available to account for the observed excess of stream yield over the measured precipitation.

Accuracy of Areal Rainfall Estimates. A question has probably arisen in the mind of almost every one having to do with areal rainfall estimates

* There should be a difference of at least 12 to 16 inches to account for water losses. For description of these basins see *Engineering News Record*, February 28, 1924, pp. 355–358. The run-off data there given are incorrect and require division by 4.0. Corrected data appear in a later issue.

as to the dependability of the results. The question also naturally arises, — What is the relative accuracy of two rainfall estimates for the same area, based on the same rainfall records but using different methods of weighting or computation? Table 9 shows the relative values of the areal mean rainfall on the Scituate basin in Rhode Island obtained by different methods of computation. The locations of rainfall stations and the rainfall contours as drawn are shown on Fig. 15. The arithmetic average was not used, but computing this from the records for the five interior stations, the result obtained is 45.83 in., which is practically identical with the results

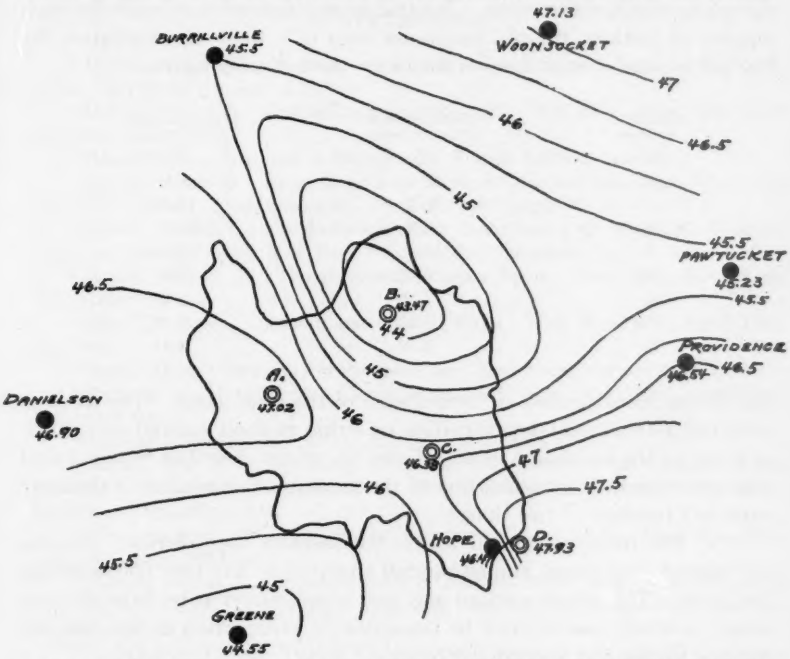

FIG. 15.— DETERMINATION OF MEAN RAINFALL ON SCITUATE BASIN BY CONTOURS.

obtained by other methods. The agreement between the long-term areal means obtained by the Thiessen, contour, and center of gravity methods respectively, is so close in this case as to afford little choice between the methods. The small range in these values is in part accounted for by the limited variation in rainfall within the drainage basin itself. The mean precipitation for the station having the lowest long-term average is 43.47 in., and for the highest long-term average 47.02 in. The data are not sufficient to show with certainty which of the methods of estimation here used is really the most reliable.

A series of 42 rain gages was maintained within an area of 48.87 sq. miles in the basin of the River Derwent in England for the 13 years 1900

to 1912. There is considerable variation in topography and also in rainfall within this area. Close spacing of the stations makes it practically certain, however, that the arithmetic average of the records represents the true areal mean rainfall. On this assumption the error resulting from calculating the rainfall from different limited numbers of stations chosen at random, using both the arithmetic averages and using the Thiessen method, has been determined (ref. 15). It was found in this case that for a limited number of stations, less than eight, the Thiessen method generally gives considerably smaller percentages of error compared with the average of the whole number of stations. Several determinations were made for each number of stations and the maximum error of a single determination for any given number of stations is shown by the following figures:

Number of stations.	Maximum error, arithmetic mean.	Maximum error, Thiessen method.
(1)	(2)	(3)
1	31.8	31.8
2	20.7	15.3
3	16.3	12.8
4	10.1	11.1
5	6.92	6.47
6	7.25	3.33
7	7.55	3.98
8	6.58	4.44

The errors are expressed in percentages of the true mean. The average error for an individual determination by either method is about one-fourth as great as the maximum error. From the study described it was found that the error in a determination of the areal mean rainfall on a drainage basin is a function of two things:

(1) The range of rainfall within the basin or the difference between the highest and lowest annual rainfall amounts at any two points within the basin. The actual stations may not be so located as to show the true range, in which case it must be estimated by comparison of the data for stations within and around the basin.

(2) It is a function of the number of rainfall records.

Formulas giving the average and maximum errors of determinations of areal rainfall by the use of the arithmetic mean, derived from the studies of the Derwent basin rainfall, are as follows, the errors being expressed in percentage of the true mean rainfall:

$$\text{Average error} = \frac{14.7\ R}{P\sqrt{N}}$$

$$\text{Maximum error} = \frac{55\ R}{P\sqrt{N}}$$

R is the range in inches; P is the mean precipitation over the area; N the number of station records used.

In Massachusetts, with one rainfall station to every 321 sq. miles, and with an average rainfall gradient of about 1 in. in 5 miles, the rainfall at a single station within any area of less than 100 sq. miles is quite certain to represent the true areal mean rainfall on the area within an error of 2 per cent, especially if the rainfall station is centrally located in the area. For larger areas in New England with the usual number of rainfall stations available an assured accuracy of the areal mean within 3 per cent is generally obtainable.

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Discussion.

Mr. Charles W. Sherman.* Mr. Horton has called attention to the fact that a change in the accuracy of the reported results may result from a change in the location or exposure of the gage. There is perhaps an equal possibility of a change in the character of the observations resulting from a change of the instrument used as a rain gage or of the observer. The personal equation of the observer is probably of particular importance in connection with the proper measurement of precipitation in the form of snow.

An illustration of the inaccuracy which may have been introduced into some old records by changes in the instruments used for collecting the rain is given by Mr. Desmond FitzGerald in a report which was reprinted in the Journal of this Association for December, 1887. He says, "The Cochituate records start from 1852 . . . but owing to a variety of causes they are untrustworthy previous to the year 1872 or thereabouts . . . In 1886 I had a letter from some one who wrote that he visited the lake during the latter part of the war and found that the gage in use had a piece of tin soldered on to the outside of the collecting rim and extending upwards for about one foot; that he impressed upon the Superintendent the inaccurate results which were being observed, etc. After a search I found this very gage in the loft of an old stable, and it is now in my custody. It added about 7 p.c. to the proper area of the gage, but I have been unable to ascertain how many years this novel device was in use."

It is entirely possible that similar "improvements" of other rain gages, the records of which have been published, may have been made, and it must always be borne in mind in using old records that there was no such thing as a standard rain gage or a standard method of setting at that time, and for that reason as well as others, the records should be scrutinized with care and tested by every available method before being accepted.

Mr. H. K. Barrows.† Mr. President, Mr. Horton has certainly directed our attention to one of the serious deficiencies in our precipitation records here in New England, — and elsewhere as well, — and that is the lack of records at the higher elevations.

The drainage areas of many of our New England streams reach elevations in excess of 2 000 ft., whereas precipitation stations at or above this elevation are but three in number, as will be noted in the following table, where is given a summary with respect to elevation of the stations listed in the records for New England by Mr. X. H. Goodnough (Journal of the New England Water Works Association, September, 1921), (including also the Mt. Washington record, at El. 6 293, kept between 1872–1886 by the United States Weather Bureau, but not now being obtained).

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* Of Metcalf & Eddy, Consulting Engineers, Boston, Mass.

† Consulting Engineer, Boston, Mass.

RANGE IN ALTITUDE OF PRECIPITATION STATIONS FOR NEW ENGLAND.

Elevation. ft.	Number of Stations.	Per cent of Total Number.
*2 000–up	3	1.0
1 500–2 000	7	2.5
1 000–1 500	23	8.5
500–1 000	59	22.
0– 500	175	66.
Totals	267	100

To determine the true precipitation on a drainage area heading into these higher altitudes is a matter requiring some study. While altitude-precipitation curves, or relations, may often be worked out and applied over limited areas, this may lead to error unless care is taken in noting the possibility of " rainfall shadows " in areas on the leeward side, as regards the prevalent moisture-laden winds, of the higher mountains and ridges.

With such large areas as the Merrimack River, for illustration, there are precipitation stations enough in *number*, but there are none of these in the higher altitudes and greater rainfalls of the Pemigewassett (as well as some of the other main tributaries of the Merrimack), so that the mean of even a considerable number of rainfall records results in too low a rainfall.

On a small area of relatively high elevation even greater errors are likely to be made in using records taken at much lower elevations, unless proper correction be made.

The Weather Bureau and other interested parties should certainly be encouraged to extend the scope of precipitation records to more adequately cover these regions of higher altitude.

Mr. Horton, as usual, has made a valuable contribution to the literature of hydrology in his paper, and has outlined some very practical methods of study that should be generally useful.

MR. R. A. HALE.† Mr. President, in connection with the run-off from the Merrimack River drainage area as measured at Lawrence, the location of rain gages in various portions of the watershed is a matter of some importance. The location of the various gages were indicated on a large map of the drainage area in New Hampshire and Massachusetts and an attempt was made to proportion the rainfall at various stations in reference to the area covered. This was found very impracticable, owing oftentimes to local conditions and other factors which entered into the question. There are about 40 such stations, and it was finally concluded to take arithmetical average as being as accurate as the conditions would warrant. The variation in one instance of rain gages five miles apart was about 4 in. in the annual amount and there was always the question as to

* Savoy, Mass. (Hoosac Mt.), El. 2 000 ft.;
Somerset, Vt., El. 2 096 ft.;
Mt. Washington, N. H., El. 6 293 ft.

† Engineer, Essex Company, Lawrence, Mass.

where the proportion should be taken. The most useful addition for accuracy would be to place additional rain gages on the drainage area where large gaps occur, and this is being done as opportunity offers.

The drainage area of the Merrimack River is about 4 452 sq. miles, over which 40 rainfall stations are scattered. The northern part of the area in New Hampshire retains the snow for a much longer period than the southern portion where the snow disappears earlier in the season.

Measurements have been made of the flow of the river at Lawrence for a period of 40 years, and records have been kept for a period of 75 years from which flows of the stream could be determined. The paper of Mr. Horton's is extremely interesting and his methods should be applied as far as possible to general conditions.

MR. CALEB M. SAVILLE.* Mr. Chairman, speaking of the variation in rainfall records, I should just like to call attention to the differences which occur in records taken with an automatic gage — the Friez recording gage, for instance — and the ordinary standard Weather Bureau gage. I was hoping that Mr. Horton would say something about the effect on rainfall from the shape of the valleys in which the gages are situated, particularly in regard to prevailing winds. We sometimes have long, narrow valleys. I am thinking particularly of the rainfall records which we are getting now at Hartford. In that area we have two valleys, — one a long, narrow valley stretching north and south, and the other a rather circular valley lying northwest and southeast, — and the gages are very close to each other, not more than five or six miles apart. Yet the rainfall, and also the run-off, as shown by the gages, in both valleys is quite a bit different, although their relative elevations are about the same and their geological characteristics also.

MR. X. H. GOODNOUGH.† Mr. President, Mr. Horton's paper is a most interesting one. I notice that the record of rainfall at Boston differed rather markedly from some of the others in a comparison of long rainfall records in Massachusetts. From such study as I have made of the Boston records, I am inclined to think that there was probably a change either in the gage itself or in its location in or about the year 1856, because from 1856 until about 1873 or thereabouts the records obtained at Boston appear to be too high. This probability is indicated by a comparison of the observations at Boston with those of other stations in the neighborhood, which show that at nearly all times the Boston gage records a higher amount than any other in the region. This condition is not true of the years previous to 1856 or subsequent to 1873.

The wide variations in the amount of rainfall collected on the watersheds of some of the carefully measured streams are probably due in part to lack of a sufficient number of rain gages or at least of gages well placed and properly maintained. The surroundings of a gage often make a great

* Chief Engineer, Board of Water Commissioners, Hartford, Conn.
† Chief Engineer, Mass. Department of Public Health.

difference in the amount of rainfall which it collects. A gage was set up at one time on the northerly end of Plum Island where it was exposed to the full force of the winds, with the result that only about 25 per cent of the rainfall was collected in this gage. By relocating the gage reasonable results were secured.

One of the most important points in the paper is the result of the study as to the probable maximum and minimum amounts of rainfall that may be expected in a given region. The determination of these limits is very important. There was a drought in England within the last few years — in 1921, I think — in which the rainfall in parts of England fell off to 70 and 60 and even 50 per cent of the average, the drought being the most intense of which there was any record for more than 200 years. There is some indication in available records in New England that a similar drought may have occurred about 160 years ago, but with that exception no evidence is yet available of greater variations than are brought out in Mr. Horton's paper.

COPPER SULPHATE TREATMENT OF CAPE POND, THE SOURCE OF WATER SUPPLY FOR ROCKPORT, MASS.

BY E. SHERMAN CHASE.*

Cape Pond, the source of public water supply for the town of Rockport, Mass., has been treated with copper sulphate during 1922 and 1923 in order to control algæ growths which give rise to the occurrence of objectionable tastes and odors. The methods employed and the results obtained are of interest, and the following account may suggest the value of more frequent adoption of this treatment for other New England water supplies affected by algæ growths.

Rockport is located at the extreme tip of Cape Ann about 30 miles northeast from Boston. The normal population of the town is about 4 000, which is increased to about 7 000 in summer, due to its popularity as a summer resort.

Cape Pond is a natural body of water located about one mile southwest from the thickly settled part of the town and about one mile west from the ocean. The ordinary high-water mark of the pond is approximately 70 ft. above mean tide. The pond has an area of about 43 acres and a maximum depth below ordinary high-water mark of 23 ft. Its capacity at ordinary high-water mark is about 182 mil. gal., these data being based upon an actual survey and soundings. The available storage capacity has been estimated at 85 mil. gal. The pond is approximately 3 200 ft. long with an average width of about 600 ft. It is one of the great ponds of the state and is, therefore, owned by the Commonwealth, from which the town has secured the right to take water for water-supply purposes.

Intake. The point of intake is approximately 900 ft. from the northerly end of the pond and approximately midway between the eastern and western shores. The intake consists of an upturned elbow surmounted by a wire screen and is located in 23 ft. of water at the deepest point of the pond. The end of the elbow is but a short distance above the bottom.

Distribution of Supply. The water is pumped from the pond to a standpipe located on a hill back of the pumping station. From this standpipe the water is distributed by gravity to the town through several miles of water mains.

Description of Pond and Watershed. Cape Pond occupies a narrow valley, the westerly and easterly slopes of which rise rather sharply from the pond to an elevation about 100 ft. above water level. There is no

* Sanitary Engineer, Metcalf & Eddy, Consulting Engineers, Boston, Mass.

inlet of any consequence, but a small intermittent outlet finds its way through a swampy area at the western shore. At the northerly end of the pond there is an area of low swampy land of 10 acres or less.

Considerable areas of shallow water, normal depth less than 10 ft., exist at the northerly and southerly ends of the pond and near the outlet at the westerly shore. Except at these points, the shores slope sharply to deep water. Considerable soft organic deposit is present in the bottom of the pond.

The fluctuations in water level vary from year to year, the maximum on record being a difference of 6.1 ft. in 1923, representing a capacity of about 78 mil. gal. The maximum high water occurs usually in the spring and the minimum in the late summer or fall. A portion of the

FIG. 1.— CAPE POND FROM SOUTH END.

stored water represented by the high level in the spring flows off through the outlet, but overflow occurs only at the higher water levels.

The island of Cape Ann is composed of three classes of material, namely, bed rock of hornblendic granite, igneous dike rock, and glacial drift deposits.* The watershed of Cape Pond is approximately 222 acres in area and consists mainly of drift deposits with a small amount of granite ledge on the hill upon which the standpipe is located. The material comprising this glacial drift has an average depth exceeding 50 ft., and at some places on the cape is known to have a depth as great as 150 ft. The material of the drift is of a fairly porous character and contains many boulders. In view of the absence of any sizable inlet, it appears probable that the pond is largely spring-fed from ground waters collected in and percolating through the glacial drift. This is of significance in considering the quality of the water and the growth of microörganisms therein.

* See "Geology of Cape Ann," by N. S. Shaler, in the ninth report, part C, U. S. Geological Survey, page 539, *et. seq.*

The watershed is of little agricultural value and with the exception of a very few acres is covered with underbrush, scrub pine and deciduous trees.

Tastes, Odors and Discoloration of the Water. The principal complaints relative to the supply have been in connection with disagreeable tastes, odors and discoloration. These unsatisfactory qualities have occurred mainly in the summer and have given rise to a feeling of dissatisfaction among the townspeople and summer visitors. Present-day standards require that a water supply shall not only be free from contamination but also attractive and wholesome.

Microscopic Organisms. The water leaching through the swamp at the northern end of the pond dissolves considerable amounts of organic matter which provides food for the microörganisms in the pond. The life, death and subsequent decay of these organisms result in the disagreeable tastes and odors. These organisms grow best in waters rich in organic matter and particularly in ground waters which contain oxidized nitrogen in the form of nitrate. Furthermore, sunlight is an essential requisite for their development.

In Cape Pond there are various types of microörganisms at different seasons and in different years. One kind predominates and then disappears to be followed by another. Examinations made by the State Department of Public Health over a period of years show that the predominating organisms have been Asterionella, Cymbella, Melosira, Synedra, and Tabellaria of the Diatom family; Anabaena and Coelosphaerium of the Cyanophyceae group; Cosmarium, Protococcus, Scenedesmus, Staurogenia and Staurastrum of the Chlorophyceae group; and Chlamydomonas, Dinobryon, Euglena, Peridinium and Trachelomonas of the Infusoria group. Of these various organisms, Asterionella, Tabellaria, Anabaena and Dinobryon appear to have occurred most frequently. Asterionella gives an aromatic, geranium or fishy odor depending upon the intensity of occurrence of these organisms. Anabaena gives a grassy and moldy odor which has been likened to that of green corn, and upon death and decay produces a very disagreeable odor resembling that of a pigpen. Dinobryon, like other protozoa, gives a fishy odor which is oftentimes one of the worst encountered in water supplies.

While odors of various kinds may be present in the supply throughout the greater portion of the year, they are more apt to be present during the summer months when the organisms upon death and decomposition liberate the essential oils, contained within their cell structures, which cause the odors. There is, however, no evidence that these microörganisms produce sickness.

Red Water. Part of the trouble with the Rockport supply has been due to corrosion of mains and services giving rise to water containing rusty sediment. This trouble has occurred mainly in the hot-water pipes and has been of more prevalence at the end of the summer. This rusty water not only offends the eye but causes rust spots on laundry.

Waters containing relatively large amounts of organic matter or of carbon dioxide are very liable to give rise to this red water difficulty. The analyses of the Rockport supply show it to be high in organic matter and soft. The few data available relative to the amount of carbon dioxide indicates that the quantity present at times is sufficient to stimulate the corrosive action of the water. This prevalence of carbon dioxide is apt to be greatest when decomposition processes are proceeding at the most rapid rate, as in the summer months when the temperatures are high.

Sanitary Analyses. The sanitary chemical analyses of this supply show a water with a turbidity ranging from very slight to distinct and with sediment varying from very slight to considerable. The color is moderately high and the residue on evaporation, particularly the organic residue, is decidedly high for an unpolluted water and is a result probably of the leaching of organic matter from the swamp at the head of the pond.

The figures for free ammonia are very variable and at times are high for an unpolluted supply. The figures for albuminoid ammonia are also relatively high and variable and are probably due principally to vegetable organic matter from the swamp and to some extent to the microörganisms in the pond water. Figures for nitrogen in the form of nitrates and nitrites are not available in recent years. The previous results, however, indicate that although the amounts of oxidized nitrogen in the pond are usually relatively small, there have been times when amounts sufficient to stimulate algæ growth have been present.

Filtration Recommended. Filtration of the supply has been recommended at various times in the past; for example, by H. W. Spooner in 1903, Henry A. Symonds in 1919, and by Metcalf & Eddy in 1922. Aëration as an adjunct to filtration has also been advised.

Temporary Remedial Measures were advised during the summer of 1922, pending the installation of a filtration plant. It was suggested, therefore, that an attempt be made to control microörganisms in the pond with copper sulphate. It was pointed out, however, that such a measure was only a temporary measure and one not to be depended upon for permanent improvement.

Survey of Pond. In order to have accurate data for proportioning the quantity of copper sulphate applied, a survey of Cape Pond with soundings was made during the latter part of May, 1922. Prior to the completion of the surveys, the numbers of microörganisms in the pond water had begun to increase and complaints relative to the quality of the supply were being received by the water works officials. In order to control these growths, it was decided to proceed with the application of the copper sulphate without waiting for the completion of the survey and soundings, using approximate estimates of the pond capacity for computing the needed dose of copper sulphate.

Application of May 22, 1922. On this date some 425 lbs. of copper sulphate, equivalent to 0.28 p.p.m. ($2\frac{1}{4}$ lbs. per mg.) were applied to Cape

Pond by towing bags containing this chemical from the stern of a boat rowed around the pond. Reasonable equalization of distribution was obtained by rowing more rapidly in shallow water and more slowly in deep water, and also by rowing more rapidly when the bag was full of copper sulphate than when it was nearly empty.

Samples of the pond water were collected prior to the application of copper sulphate, one being taken over the intake, the second about halfway between the intake and the southern end of the pond, and the third sample about halfway between the intake and the pumping station at the northern end of the pond. All three samples were collected at points about 1 ft. below the surface. The samples had a slight musty and fishy taste and contained distinct amounts of fine organic suspended matter.

On May 24, two days after the application of copper sulphate, samples were collected at practically the same points as those collected on May 22. The water in the pond was still distinctly turbid with what were evidently growths of organisms. The samples were examined at the laboratory of the State Department of Public Health and the total numbers of organisms, numbers of genera and numbers of principal organisms found are given in the following table (Table 1).

This table shows that there was comparatively little difference in the total numbers of organisms or in the predominating types.

The samples collected May 24 showed somewhat higher total counts than those collected on May 22, due mainly to an approximately 100 per cent increase in the numbers of Anabaena. Asterionella, however, were materially reduced and organisms of the protozoan types were practically eliminated. Subsequent samples, collected on May 26, showed a total number of organisms less than one-half those present in the samples of May 22, and Anabaena about one-fourth the number present in the samples of May 24.

This first application of copper sulphate had no effect upon the fish life of the pond, and although the turbidity of the surface water continued to be distinct, that drawn from taps in the village had become satisfactory in so far as appearance and tastes were concerned.

Application of May 27, 1922. In view of the failure of the copper sulphate to bring about a greater decrease in microscopic organisms, and in view of the relatively small amount of copper sulphate first used, it was considered advisable to make another application, which was done on May 27 in a manner similar to that of May 22. The total amount of copper sulphate applied at the time of this second application was about 920 lbs. equivalent to 0.6 p.p.m. (5 lbs. per mg.).

The samples collected on May 26 represented the condition of the water prior to this second treatment. Samples were collected on May 29 and others on June 8, 12 days later. As in the case of the samples collected 2 days subsequent to the first application, the samples of May 29 after the second application showed a distinct increase in total numbers of

TABLE 1.

RESULTS OF COPPER SULPHATE TREATMENT AS SHOWN BY REDUCTION IN CERTAIN PRINCIPAL AND TOTAL MICROSCOPIC ORGANISMS PRESENT IN CAPE POND WATER. MAY APPLICATION.

Sample Cape Pond.	Date 1922.	Diatomaceae.			Cyanophycae.	Infusoria.		Total Organisms.	Total Genera.
		Asterionella.	Synedra.	Tabellaria.	Anabaena.	Monas.	Synura.		
Before Copper Sulphate Treatment									
North End	May 22	68	4	86	226	6	76	684	20
South End	22	88	6	26	298	0	58	586	17
Intake	22	112	2	20	244	0	94	612	13
After First Treatment of May 22									
North End	24	44	8	42	542	6	0	690	13
South End	24	84	16	14	472	0	0	680	17
Intake	24	44	12	46	476	0	0	624	10
North End	26	48	4	24	158	4	0	326	15
South End	26	12	20	10	126	2	0	244	13
Intake	26	44	2	14	90	0	0	230	11
After Second Treatment of May 27									
North End	29	40	10	2	320	0	0	510	12
South End	29	74	0	6	346	2	0	526	10
Intake	29	112	8	22	340	0	0	616	10
East of Intake	June 8	0	1	1	0	1	0	40	7
West of Intake	8	0	0	0	8	4	0	62	4
Intake	8	46	6	2	0	4	0	120	7

May 22. 425 lbs. copper sulphate, 0.28 p.p.m., applied.
May 27. 920 lbs. copper sulphate, 0.60 p.p.m., applied.

organisms, but at this time, Asterionella increased as well as Anabaena. The samples collected June 8, however, showed a marked reduction in total numbers of organisms, and Asterionella and Anabaena were practically absent.

It is interesting to note that in both cases the treatment was not followed by the immediate destruction of the organisms. It appears probable that this retardation of effect was due to the time required for the diffusion of the copper sulphate throughout the water. Had it been feasible to row the boat in lines closer to each other, the original application would have been more uniform and more rapid results would probably have been secured. It is probable also, that the effect would have been obtained more promptly had the chemical been dissolved in water and sprayed over the surface of the pond; but this would have been somewhat more difficult and would have involved greater expense.

The second application followed the first so closely that the accumulated quantity of copper sulphate in the pond water was equivalent to about 0.88 p.p.m. ($7\frac{1}{4}$ lbs. per million gallons). This amount of copper sulphate not only resulted in the destruction of the majority of the microorganisms but also brought about the death of considerable numbers of white perch. The first dead fish were noticed on Sunday, May 28, the day following the second application. The record of fish killed is shown in the following tabulation (Table 2):

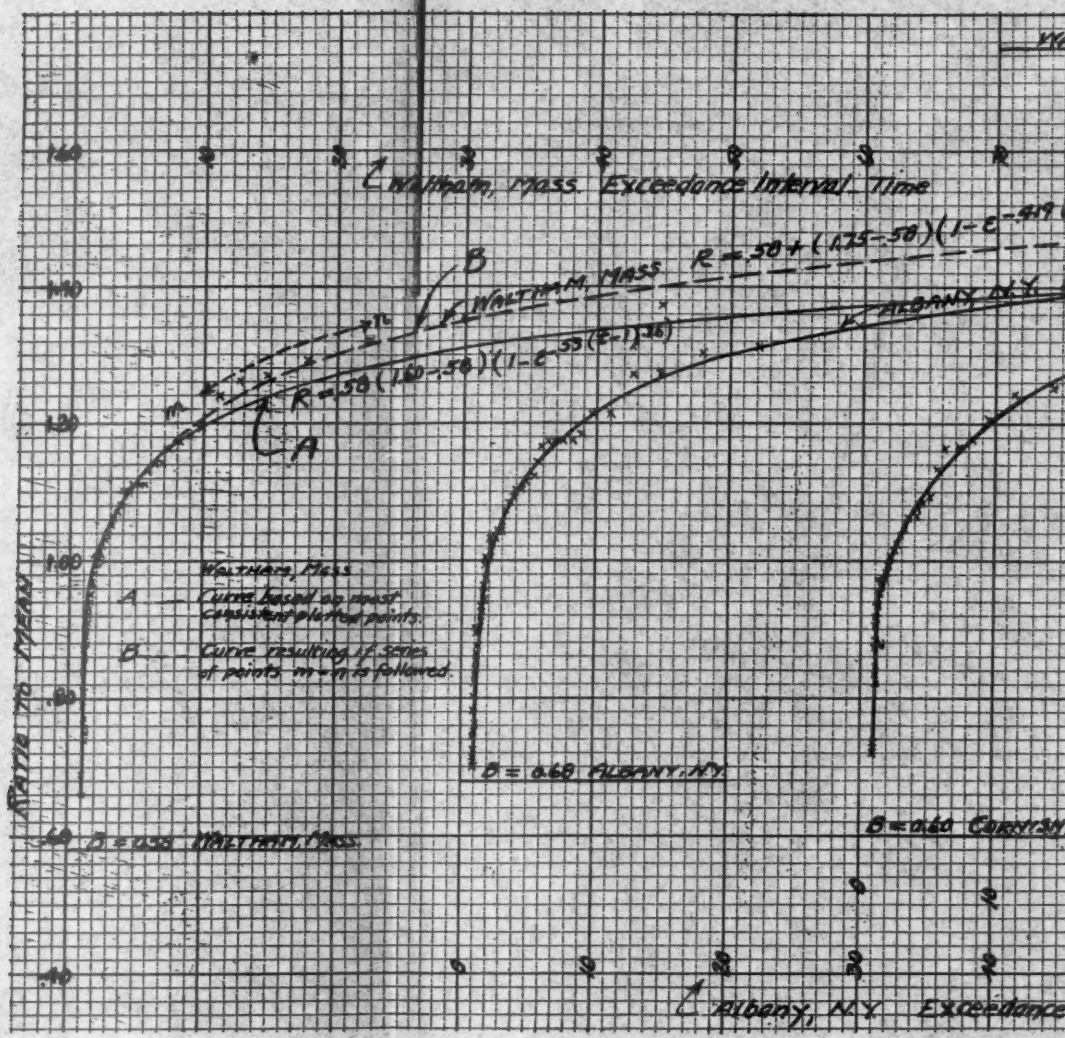
TABLE 2.

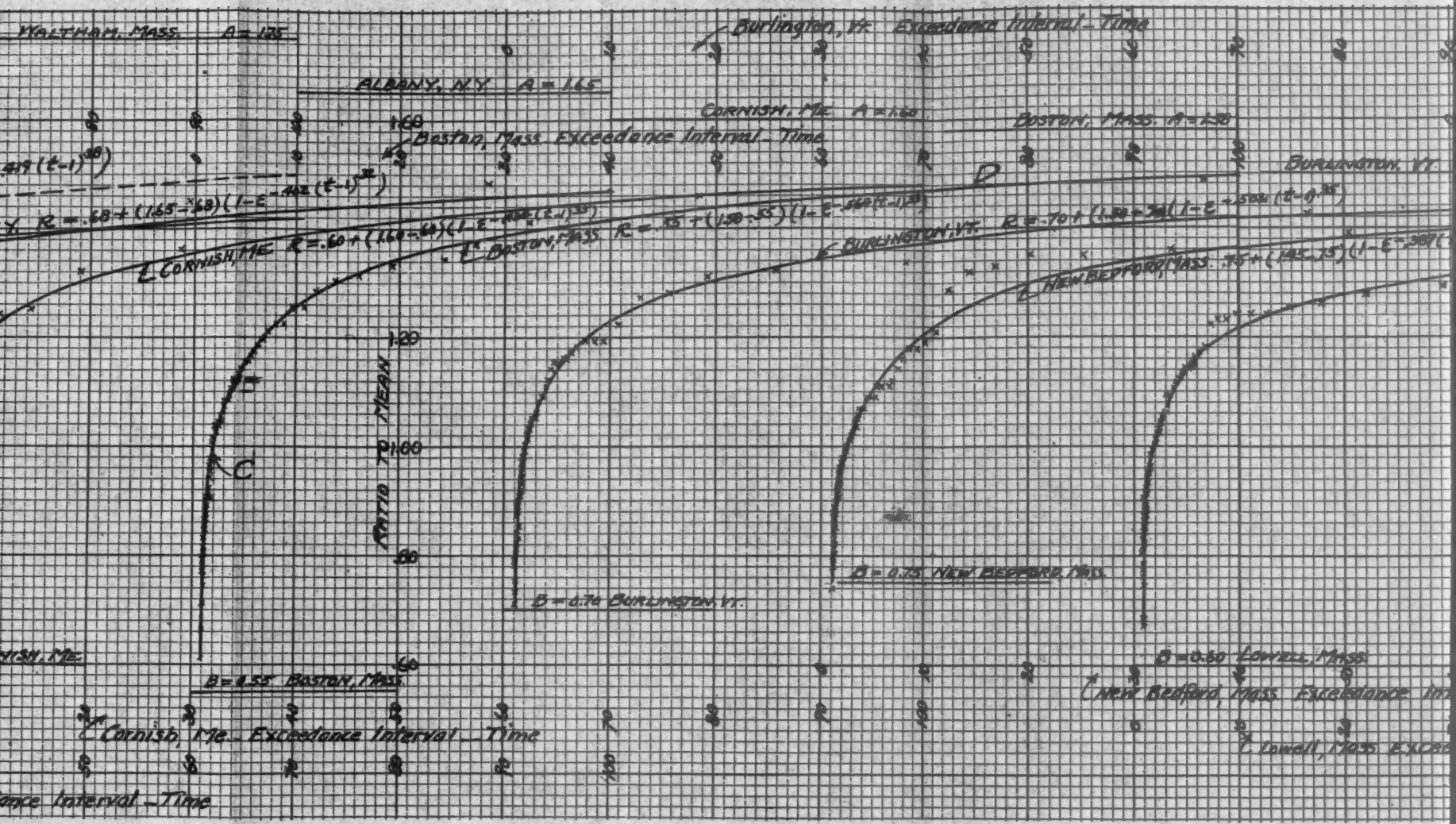
DATE RELATIVE TO DEAD FISH REMOVED FROM CAPE POND.

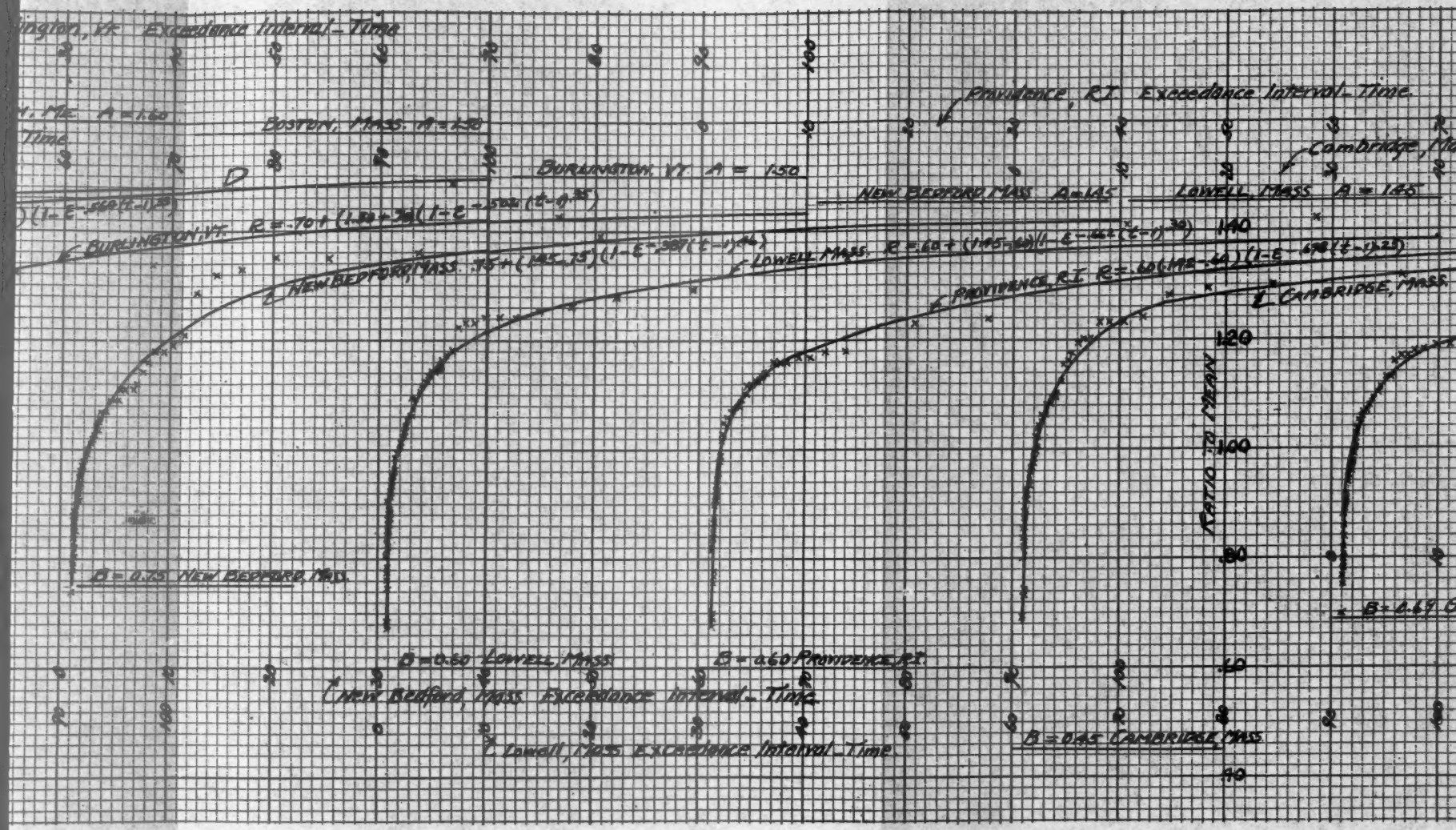
Sunday, May 28	22 large fish
Monday, May 29	195 large fish about 10 in. long
	305 small fish 4–5 in. long
Tuesday, May 30	155 large fish 8–12 in. long
	410 small fish 2–6 in. long
Wednesday, May 31	276 large fish
	208 small fish
Thursday, June 1	55 large fish 6–10 in. long
	110 small fish 3–6 in. long
Monday, June 5	60 large fish
	40 small fish

No fish removed after June 5.

The fish killed consisted exclusively of white perch, although fish of other species are said to be present in the pond. According to data given in Whipple's "Microscopy of Drinking Water," the limit of copper sulphate application to avoid killing perch is 0.75 p.p.m., an amount exceeded by about 0.1 p.p.m. by the combined applications of May 22 and May 27. Inasmuch as the dead fish were removed from the pond daily, no harm resulted from their deaths; in fact, the destruction of such numbers of fish may have discouraged fishermen from visiting the pond and in this way minimized in some extent the danger of contamination. It should


Waltham, Mass. Exceedance Interval Time
R = 50 + (175 - 50)(1 - e^-.419
B
Waltham, Mass
R = 50 (1.60 - .50)(1 - e^-.55(t-1).36)
A
Albany N.Y.
Waltham, Mass.
A — Curve based on most consistent plotted points.
B — Curve resulting if series of points m + n is followed.
Ratio to Mean
D = 0.60 Albany, N.Y.
D = 0.60 Waltham, Mass.
D = 0.60 Cohasset
Albany, N.Y. Exceedance


WALTHAM, MASS. A = 1.75
ALBANY, N.Y. A = 1.65
CORNISH, ME. A = 1.60
BOSTON, MASS. A = 1.50
BURLINGTON, VT.
Burlington, Vt. Exceedance Interval - Time
Boston, Mass. Exceedance Interval - Time
CORNISH, ME.
BOSTON, MASS.
BURLINGTON, VT.
NEW BEDFORD, MASS.
RATIO TO MEAN
1.60
1.20
1.00
.80
.60
B = 0.70 BURLINGTON, VT.
B = 0.55 BOSTON, MASS.
D = 0.75 NEW BEDFORD, MASS.
D = 0.60 LOWELL, MASS.
Cornish, Me. - Exceedance Interval - Time
New Bedford, Mass. Exceedance Int
Lowell, Mass. Exced


Burlington, VT Exceedance Interval - Time
Boston, Mass. A = 1.50
Burlington, VT A = 1.50
Burlington, VT R = 70 + (1.50 - 70)(1 - ε^-.550(t-1)^.70)
New Bedford, Mass. R = 75 + (1.45 - .75)(1 - ε^-.38(t-1)^.44)
B = 0.75 New Bedford, Mass.
New Bedford, Mass Exceedance Interval - Time
Providence, R.I. Exceedance Interval - Time
New Bedford, Mass. A = 1.45
Lowell, Mass. A = 1.45
Lowell, Mass. R = 60 + (1.45 - .60)(1 - ε^-.46(t-1)^.30)
B = 0.60 Lowell, Mass.
Lowell, Mass Exceedance Interval - Time
Providence, R.I. R = 60 + (1.45 - .60)(1 - ε^-.68(t-1)^.25)
B = 0.60 Providence, R.I.
Cambridge, Mass.
B = 0.67 Cambridge, Mass.
Ratio to Mean
140
120
100
80
60
40

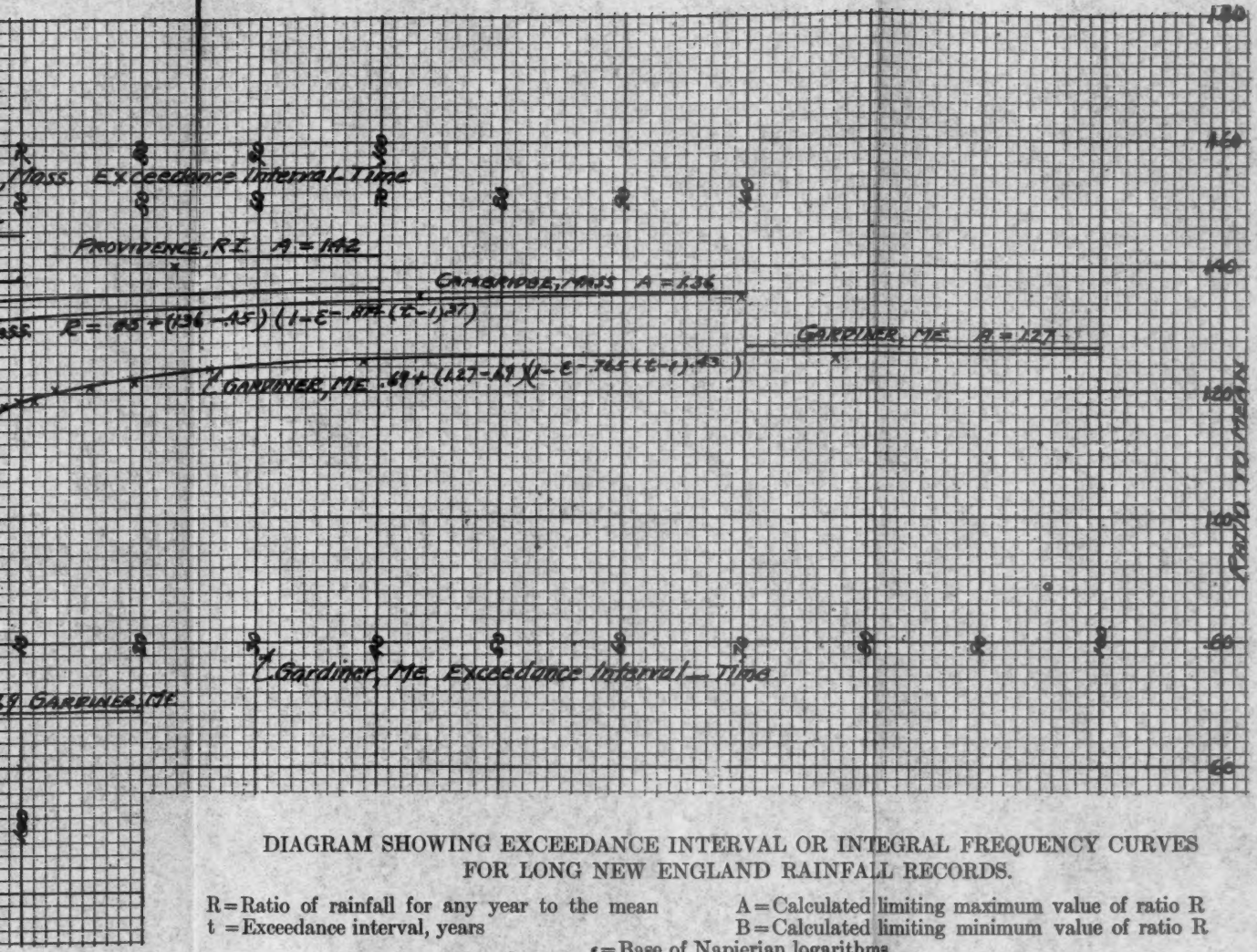
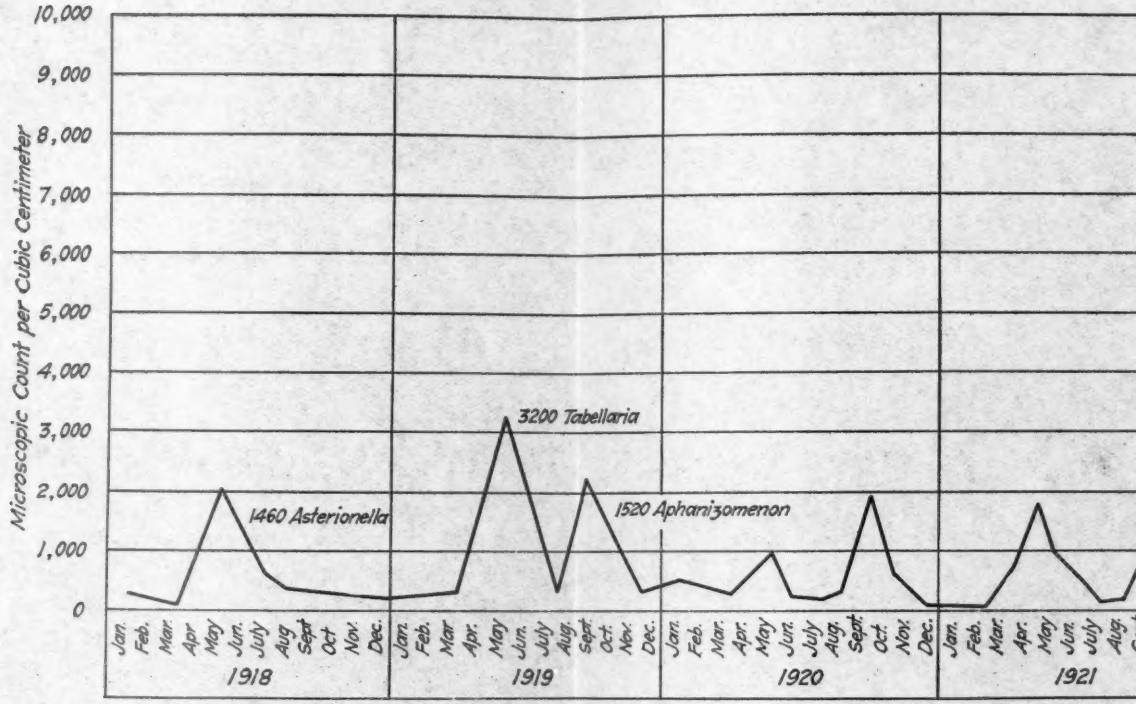


DIAGRAM SHOWING EXCEEDANCE INTERVAL OR INTEGRAL FREQUENCY CURVES FOR LONG NEW ENGLAND RAINFALL RECORDS.

R = Ratio of rainfall for any year to the mean
t = Exceedance interval, years
A = Calculated limiting maximum value of ratio R
B = Calculated limiting minimum value of ratio R
ϵ = Base of Napierian logarithms

Note:- Tap samples only, 1918-1921 incl.
Average of all samples, 1922 and 1923.



TOTAL NUMBER OF MICROSCOPIC ORG
R

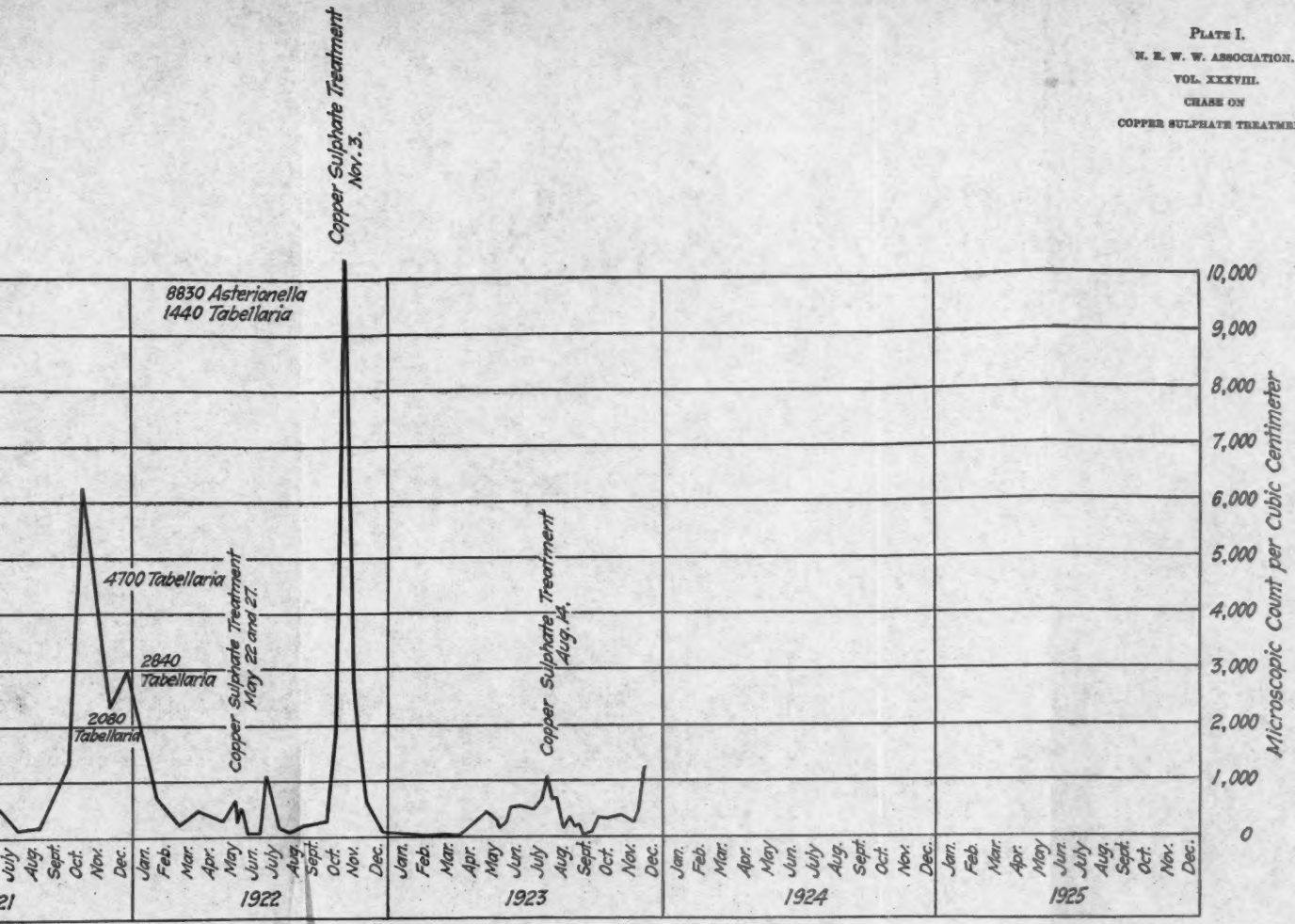
PLATE I.

N. E. W. W. ASSOCIATION.

VOL. XXXVIII.

CHASE ON

COPPER SULPHATE TREATMENT.



ORGANISMS IN CAPE POND WATER (1918–1923 INCLUSIVE),
ROCKPORT, MASS.

be noted that these first two applications were made prior to the availability of accurate data relative to the capacity of the pond, and for this reason the best proportioning of the dose could not be secured.

Throughout the summer of 1922 frequent samples of the supply were collected and microscopical examinations were made by the State Department of Public Health. For the purposes of comparison the total counts of microscopic organisms in these samples and in the samples obtained during the past six years have been plotted on the accompanying diagram.

From this diagram several things are apparent. First, there have usually been two periods of high counts during the year, the first period occurring in May and the second in the fall, about October. Second, the predominating organisms at these times of high counts have been of the Diatom class, organisms which give aromatic odors. Third, during 1922 the spring peak was kept low, apparently by the application of copper sulphate in May. The small peak in July, 1922, was due to the presence of Monas, an organism of the protozoan type which has not ordinarily occurred in the supply in large numbers. This prevalence of Monas was evidently the effect of the copper sulphate treatment producing the right environmental conditions for this type of organisms.

Application of November 3, 1922. The counts of microscopical organisms remained low during the summer and early fall. Samples collected during August, however, showing a marked increase in total numbers, principally Diatoms. As the numbers of organisms appeared to be increasing, it was considered advisable to again treat the pond with copper sulphate and such application was made on November 3. Some 700 lbs. of copper sulphate were used equivalent to 0.5 p.p.m. (41 lbs. per mg.).

Samples were collected on November 3 prior to the application of chemical and numerous other samples were collected at intervals subsequent to this date. A summary of the microscopic analyses of these samples are shown in the following table (Table 3).

It will be noted from this table that the effect of the copper sulphate was not so marked as in the case of the second application of May 27. However, the November application resulted first in checking and finally in materially reducing the growths so that no further tastes and odors were experienced subsequent to the treatment.

Treatment of August 14, 1923. Frequent microscopic analyses of the supply were continued throughout 1923. The gradual increase in prevalence of organisms during the summer led to the application of 838 lbs. of copper sulphate to the pond on August 14. This quantity was equivalent to 0.71 p.p.m. by weight or slightly less then 6 lbs. per million gallons.

The results of microscopic examinations of samples of the pond water collected during the periods immediately before and immediately after the treatment are given in the following table (Table 4).

This table shows a distinct reduction in the numbers of organisms present in the pond water following the treatment. The reduction in

TABLE 3.

RESULTS OF COPPER SULPHATE TREATMENT AS SHOWN BY REDUCTION IN CERTAIN PRINCIPAL AND TOTAL MICROSCOPIC ORGANISMS PRESENT IN CAPE POND WATER. NOVEMBER APPLICATION.

Sample Cape Pond.	Date 1922.	Diatomaceae			Cyanophycae.	Infusoria.		Total Organisms.	Total Genera.
		Asterionella.	Synedra.	Tabellaria.	Anabaena.	Monas.	Synura.		
Before Copper Sulphate Treatment									
N. E. of Intake	Nov. 3	8 160	0	1720	0	0	0	10 070	12
S. W. of Intake	3	8 720	0	1240	0	0	0	10 088	12
Intake	3	9 600	0	1360	0	0	0	11 096	8
After Copper Sulphate Treatment									
Pond	7	6 000	0	672	0	0	0	6 752	10
Tap	7	4 000	0	210	0	0	0	4 284	7
N. W. End	10	4 800	0	380	0	2	0	5 214	8
S. W. End	10	4 560	0	344	0	0	0	4 936	7
Intake	10	4 400	4	384	0	0	0	4 830	8
Intake	16	2 560	0	120	0	0	0	2 698	6
No. of Intake	16	2 920	0	144	0	0	0	3 100	9
N. E. of Intake	27	1 536	0	84	0	0	0	1 696	10
S. W. of Intake	27	1 540	0	64	0	0	0	1 642	4
Intake	27	1 024	0	24	0	0	0	1 106	6
Pond	Dec. 4	664	0	28	0	0	0	712	5
Tap	4	504	0	40	0	0	0	580	6
Pond	26	0	1	0	0	0	0	17	4
Tap	26	24	0	15	0	0	0	48	5

November 3. 700 lbs. copper sulphate, 0.5 p.p.m., applied.

TABLE 4.

RESULTS OF COPPER SULPHATE TREATMENT AS SHOWN BY REDUCTION IN CERTAIN PRINCIPAL AND TOTAL MICROSCOPIC ORGANISMS PRESENT IN CAPE POND WATER. AUGUST, 1923, APPLICATION.

Sample Cape Pond.	Date 1923.	Diatoms.	Blue Green Algæ (Cyanophyceae).	Green Algæ.	Infusoria.	Total Organisms.	Total Genera.
			Before Copper Sulphate Treatment				
North End	July 30	908	38	42	36	1 028	12
South End	30	1 036	32	82	16	1 166	10
Intake	30	878	22	52	10	962	8
North End	Aug. 6	692	48	4	4	754	10
South End	6	392	48	2	6	448	5
Intake	6	710	66	2	22	800	8
North End	Aug. 13	492	48	102	18	660	7
South End	13	672	56	14	20	762	8
Intake	13	528	64	26	14	632	7
			After Copper Sulphate Treatment of August 14				
North End	Aug. 20	126	18	14	6	164	9
South End	20	186	10	12	0	210	7
Intake	20	170	2	8	2	180	4
Tap	Aug. 29	14	68	64	130	278	10
Cape Pond	29	8	104	104	152	368	10
North End	Sept. 4	6	38	50	86	182	10
South End	4	2	30	62	116	210	9
Intake	4	4	6	40	116	168	9

August 14. 838 lbs. copper sulphate, 0.71 p.p.m., applied.

the numbers of Diatoms was particularly marked, although there appeared a slight increase in the numbers of green algæ and infusoria as shown in the samples of August 29 and September 4. Samples collected somewhat later, on September 18 and 24, showed very low counts with respect to all types of organisms.

As in the case of the applications of 1922, the fullest effect of the treatment did not occur until some time subsequent to the date of application, due probably to the somewhat slow diffusion of copper sulphate throughout the entire body of water constituting the pond.

As previously mentioned, the second application of copper sulphate in 1922 was followed by the destruction of considerable numbers of white perch. In 1923 no fatality occurred among the larger fish. Numerous minnows, however, inhabiting the shallow portions of the pond were killed.

Referring again to the diagram showing the total counts of organisms of the past 6 years, it will be seen that on only two occasions in 1923 did the numbers of organisms exceed 1 000 per cubic centimeter. At these two times the predominating organisms were Tabellaria and Asterionella, organisms which do not cause unpleasant tastes and odors unless in excessive numbers. It is also interesting to note that the total numbers of organisms present during 1923 have been maintained on the average, much lower than during any of the previous 5 years. It is also evident from the diagram that the copper sulphate application of August 14, 1923, checked the tendency for increase in organisms. The experience of 1923 indicates that if the numbers of organisms can be kept below 1 000 per cubic centimeter, there will be comparatively little trouble from tastes and odors from such growths. It is true, however, that smaller numbers of certain organisms might give rise to trouble, but Cape Pond does not seem to be subject to these particular organisms.

Cost of the Copper Sulphate Treatment exclusive of engineering services was about $160 during 1922, made up of the cost of 2 045 lb. of copper sulphate at $6\frac{3}{4}$ cents delivered, and of the service of one boatman on two days and of two boatmen on the third day. In 1923, the cost exclusive of engineering was about $65, made up of the cost of 838 lb. of copper sulphate and the service of two boatmen for one day.

Flushing of Water Mains. During the latter part or July, 1922, complaints were received relative to the occurrence of a distinct odor and a brownish sediment in the water. These complaints were fairly well distributed over the town. At that time, the water in the pond itself contained some odor and sediment, but distinctly less than that found in the water in the town. Inasmuch as the numbers and types of microorganisms present were not such as to account for this trouble, it appeared probable that this odor was the result of decomposition of the organic matter present in the water, this decomposition going on partly in the pond and partly in the distribution system. It also appeared probable that the brownish sediment was due to iron rust formed by the action of the carbon

dioxide upon the iron pipes of the distribution system, this carbon dioxide resulting from the decomposition of the organic matter. It was suggested, therefore, that a systematic program of flushing dead ends of the distribution system be carried out.

The first systematic flushing of the mains was put into effect upon the first of August, 1922, and a second flushing about the first of November, shortly after the application of copper sulphate of November 3. At the time of the August flushing, considerable quantities of sediment were flushed out of certain hydrants, particularly those hydrants at the ends of the long mains. Following both flushings in 1922, there was a marked improvement in the physical quality of the water delivered to the consumers.

In 1923 similar flushing of the mains was put into effect twice during the year, once in May and again in August. Such action appears to have prevented undue accumulation of sediment.

Results Obtained. It is stated by C. A. Wetherbee, superintendent of the water works at Rockport, that in no prior year has there been so few complaints relative to the quality of the water as during 1923. There seems to be no doubt but that the control of the algæ growths by copper sulphate and the systematic flushing of the mains have given Rockport a supply of reasonably satisfactory physical quality, and of a quality distinctly superior to that of other years prior to the introduction of the remedial measures.

Summary.

This experience shows that the application of suitable amounts of copper sulphate in May, 1922, so checked microscopic growths as to keep their numbers materially less during the summer and early fall than during any summer prior to that year. The amount of copper sulphate applied in May was sufficient to kill considerable numbers of fish, although when the size of the pond and probable numbers of fish therein are taken into consideration it does not appear that the killing of the fish was a matter about which to be in any wise alarmed.

It is evident that the effect of copper sulphate wore off about the middle of the fall of 1922 and as usual the fall increase in microörganisms occurred, if anything somewhat more pronounced than in earlier years. It also appears evident that the amount of copper sulphate required to effectually destroy the growths present in Cape Pond has been between 0.5 p.p.m. and 0.9, p.p.m., about 0.7 p.p.m. probably being sufficient.

The occurrence of trouble with odors and sediment in the latter part of July, 1922, was probably due to the decomposition of organic matter and the corrosion of water pipes by free carbon dioxide resulting from this decomposition. Systematic flushing of the mains through hydrants and blow-offs brought about a marked improvement in the physical quality of the supply.

In 1923 the program of water main flushing and of copper sulphate treatment resulted in maintaining the physical quality of the supply in a condition superior to that obtained in any previous year.

Acknowledgment of assistance and coöperation is due T. T. Hunter Harwood, Chairman of the Special Water Committee of Rockport, Charles A. Wetherbee, Superintendent of Water, Rockport, A. D. Weston, State Department of Health, and Charles W. Sherman, past president of this Association.

Discussion.

Mr. Robert E. Horton.* Mr. President, I would like to ask Mr. Chase if there was any variation discovered in the distribution of organisms at different depths?

Mr. Chase. We made no study of depth distribution of organisms except in the samples as collected at the surface, perhaps one foot below the surface, and analyzed. Samples were also taken from taps in the town, which received water from greater depths in the pond. There did not seem to be any marked difference in types of organisms present.

Mr. Horton. How deep below the surface were the bags of copper sulphate suspended in rowing?

Mr. Chase. About three or four feet. When the bags were empty they were nearer the surface; when they were full they would sink down three or four feet.

Mr. Horton. Would you be likely to get more effective treatment in a deep pond by using two bags, one at a slight depth and the other at a comparatively greater depth?

Mr. Chase. That might be true. As a general thing, copper sulphate tends to diffuse and sink to the bottom.

Prof. George C. Whipple.† Mr. President, I thought Mr. Chase's paper an interesting one. It brings back somewhat very tender recollections of some old work I did some 25 years ago in connection with the algæ. I must say I am a good deal disappointed that science has not advanced more than it has during that period. About the only thing we have done is to learn how to kill these organisms with copper sulphate. We have not progressed as fast as we should have progressed in modes of preventing their growth. I still live in hopes that some young man will take up that subject and work at it for five or ten years until he arrives at the ultimate solution. We have made perhaps several hundred thousand microscopic analyses showing the kinds of organisms in the water, but we are not much wiser than we were 30 or 40 years ago.

Mr. Henry A. Symonds.‡ This problem of how to eradicate so-called algæ is certainly a difficult one to solve in a perfectly satisfactory

* Hydraulic Engineer, Voorheesville, N. Y.
† Professor of Sanitary Engineering, Harvard University.
‡ Consulting Civil Engineer, Boston, Mass.

manner, but the method described by Mr. Chase appears to have given excellent results at small cost.

I recall a case of algæ trouble which we had in Stockbridge. Copper sulphate was considered but not used. As illustrating the difference in the intensity of growth in different depths of the water, the lower end of the lake from which the water is taken at Stockbridge is quite shallow, and the lake as a whole for many years has contained a great deal of growth of this kind. A few years ago we built a dike across the lower end, shutting off a section of perhaps two acres. The level of the pond was raised six inches minimum above the lower end so that no flow could get back into the main lake. This was something of an experiment, but the results were remarkably good. I have not the latest reports from there, but for some time at least after this remedy was tried, the water remained substantially clear of algæ, indicating a great difference in the amount of growth which had taken place in deeper water and in shallow water.

MR. M. C. WHIPPLE.* Mr. President, I am very glad to hear this paper because it emphasizes a method of treatment which has been available for a good many years and one which has not been utilized to the extent it should have been. Many towns will benefit from it in the future if the advantages and limitations are more widely discussed. It is very cheap and very simple, and Mr. Chase has shown that the results are very striking.

There are one or two points he has mentioned which are perhaps disadvantages or drawbacks to the treatment. One is the difficulty of applying the copper sulphate in the most effective way. There is a chance for considerable ingenuity to be applied in carrying out this treatment, and there is no doubt about its being more effective if the copper sulphate can be distributed uniformly throughout the water treated.

He has also indicated that the treatment gives no assurance of lack of trouble in the future. The height of the trouble came in November, and he indicated that the copper treatment needs to be repeated at intervals. In fact, it has sometimes been found that the application of the copper sulphate results after a very short period in a very rapid increase in other types of organisms which were not present at the time of application; that is, the removal of one or more species may make way for the rapid growth of others. Mr. Chase's chart would seem to show that there were fewer organisms in the year 1923, possibly from the result of treatment in the fall of 1922. The late fall treatment no doubt served to kill off many of the cells which either survive during the winter or else give rise to spores which carry over for the next season's growth. On the other hand, it has been a matter of observation with me that the past season was not a bad one for algæ growths. Several bodies of water which I have had occasion to watch did not during this past season show as heavy growths as in some other seasons. What causes these seasonal differences it is hard to say. Pro-

* Instructor in Sanitary Chemistry, Harvard University.

fessor Whipple has indicated that our knowledge of some of these things has not increased as we had hoped for.

MR. ARTHUR L. GAMMAGE.* Mr. Chase has performed a real service to sanitary engineers and water works men by bringing to their attention a plan of control of the causes of odors and tastes in a typical water supply. It is the writer's opinion that a larger dose of copper sulphate can be added to a pond or reservoir or feeder brook with less killing of fish if the chemical is added in a solution and the treatment conducted on a windy day, when there will be a maximum mixing of chemical and water supply. We have used it this way in well-stocked ponds without killing any fish.

If algæ are made resistant to copper sulphate by continual applications, I suggest, as we have successfully used, bleaching powder treatment after copper sulphate to decompose the algæ besides killing them. Chlorine from bleaching powder or liquid chlorine will destroy many odors and tastes resulting from growing and decomposing algæ, and also many odors and tastes resulting from decomposition of organic matter under conditions of insufficient oxygen in the water, if used in the proper way.

Systematic flushing of pipes for loosely deposited iron and organic matter has improved many waters. Carbonic acid naturally in a water or coming from decomposition with low alkalinity has caused much corrosion of pipes and the rusty water appears to be iron hydrate with carbonic acid and more or less organic matter. Aëration or soda ash or both appear to be the cheapest remedy for this trouble without increasing the hardness of the water.

* Chemist, Weston & Sampson, Consulting Engineers, Boston, Mass.

LATEST DEVELOPMENTS IN CHLORINE CONTROL APPARATUS FOR STERILIZATION OF WATER SUPPLIES.

BY GILBERT H. PRATT.*

[*Read September 21, 1923.*]

Sterilization of water supplies by means of liquid chlorine is one of the outstanding advances in sanitary science in the past ten years.

Chlorination of water supplies has probably been the largest contributor in reducing typhoid fever death rates to the point where they show figures below the most optimistic estimates of sanitarians of only a few years back.

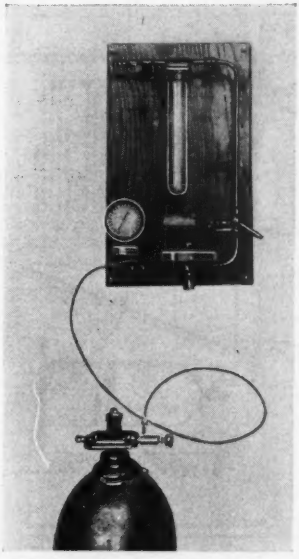


FIG. 1. — FIRST CHLORINATOR.

Fig. 1 shows the first chlorinator which was installed in the spring of 1913 by Mr. Tiernan at Dover, N. J., in connection with protecting the quality of the water supply of Jersey City, N. J. Compared with those which have followed, we cannot fail to be impressed with the improvements made.

The machine shown in Fig. 2 represents the up-to-date smallest capacity chlorinator for applying chlorine solution to pump suctions or

* New England Representative, Wallace & Tiernan Company, Newark, N. J.

open bodies of water, where a water supply is available under at least fifteen pounds pressure to operate the apparatus. The flow of gas is measured by means of a volumetric type of meter.

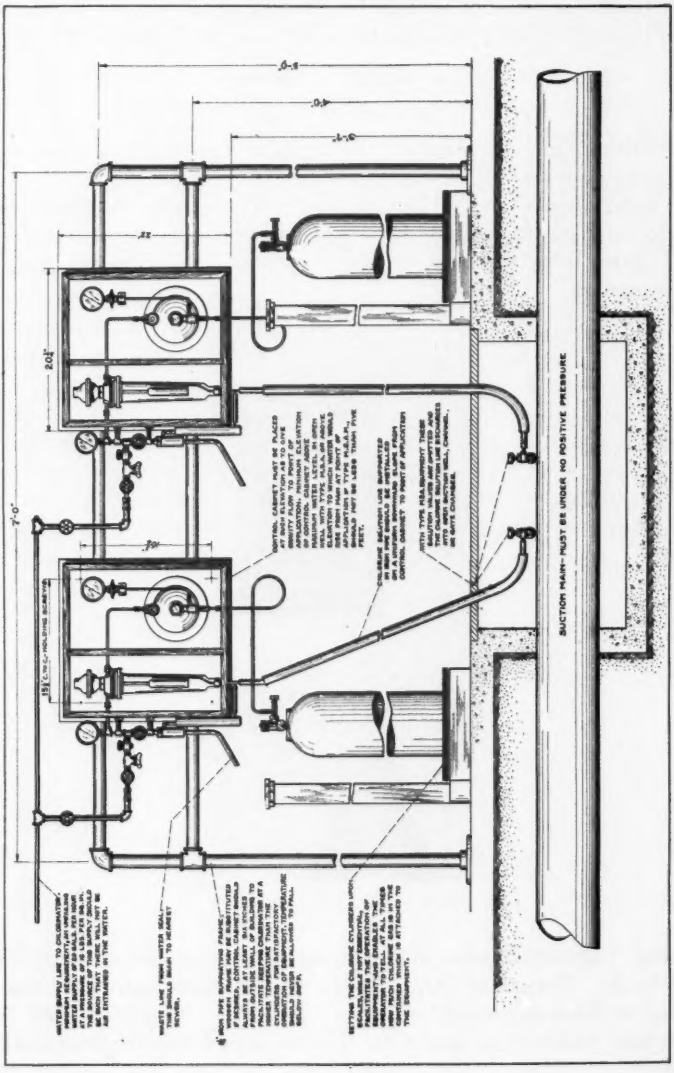


FIG. 2. — TYPE MSAM.

A duplicate installation of machines is shown. This duplication of mechanical equipment, so that at all times service may be an absolute certainty, is in line with good engineering practice.

One adaptation of this equipment has been for the sterilization of swimming pool recirculating systems. Usually in such installations a slight back pressure exists, due to friction losses in the pipe line from the filters to the pool, and in order to overcome this back pressure which would exist,

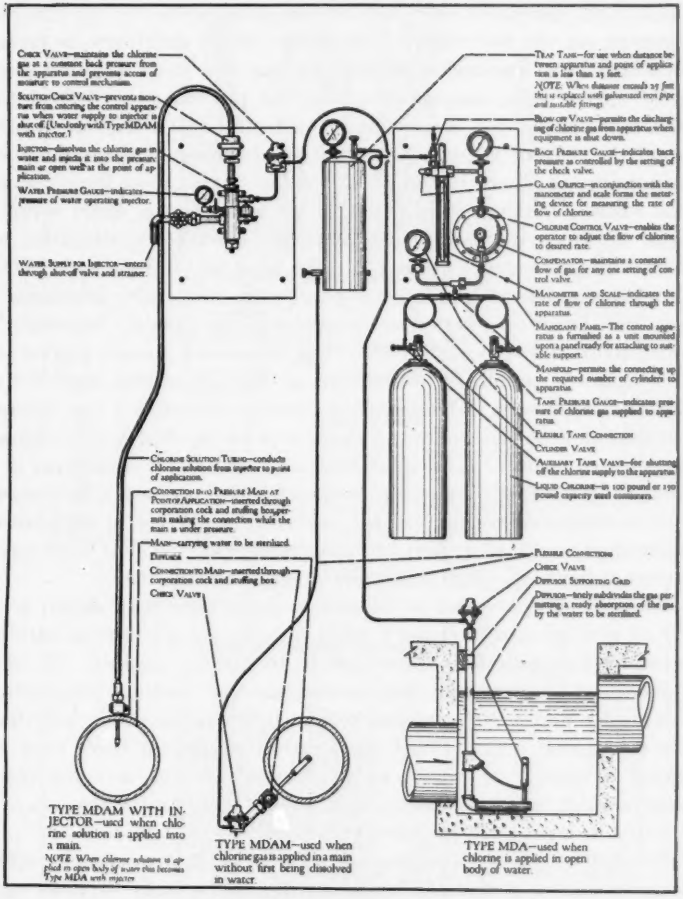

Check Valve—maintains the chlorine gas at a constant back pressure from the apparatus and prevents access of moisture to control mechanism.
Solution Check Valve—prevents moisture from entering the control apparatus when water supply to injector is shut-off. (Used only with Type MDAM with injector.)
Injector—dissolves the chlorine gas in water and injects it into the pressure main or open well at the point of application.
Water Pressure Gauge—indicates pressure of water operating injector.
Water Supply for Injector—enters through shut-off valve and strainer.
Chlorine Solution Tubing—conducts chlorine solution from injector to point of application.
Connection to Pressure Main at Point of Application—inserted through corporation cock and stuffing box, permits making the connection while the main is under pressure.
Main—carrying water to be sterilized.
Diffuser
Connection to Main—inserted through corporation cock and stuffing box.
Check Valve
Trap Tank—for use when distance between apparatus and point of application is less than 25 feet.
NOTE. When distance exceeds 25 feet this is replaced with galvanized iron pipe and suitable fittings.
Blow-off Valve—permits the discharging of chlorine gas from apparatus when equipment is shut down.
Back Pressure Gauge—indicates back pressure as controlled by the setting of the check valve.
Glass Orifice—in conjunction with the manometer and scale forms the metering device for measuring the rate of flow of chlorine.
Chlorine Control Valve—enables the operator to adjust the flow of chlorine to desired rate.
Compensator—maintains a constant flow of gas for any one setting of control valve.
Manometer and Scale—indicates the rate of flow of chlorine through the apparatus.
Mahogany Panel—The control apparatus is furnished as a unit mounted upon a panel ready for attaching to suitable support.
Manifold—permits the connecting up the required number of cylinders to apparatus.
Tank Pressure Gauge—indicates pressure of chlorine gas supplied to apparatus.
Flexible Tank Connection
Cylinder Valve
Auxiliary Tank Valve—for shutting off the chlorine supply to the apparatus.
Liquid Chlorine—in 100 pound or 150 pound capacity steel containers.
Flexible Connections
Check Valve
Diffuser Supporting Grid
Diffuser—finely subdivides the gas permitting a ready absorption of the gas by the water to be sterilized.
TYPE MDAM WITH INJECTOR—used when chlorine solution is applied into a main.
NOTE. When chlorine solution is applied in open body of water this becomes Type MDA with injector.
TYPE MDAM—used when chlorine gas is applied in a main without first being dissolved in water.
TYPE MDA—used when chlorine is applied in open body of water.

FIG. 3. — TYPES MDA AND MDAM.

an auxiliary injector is installed below the machine. This injector is operated by a pressure water supply, the requirements usually being three to four times the pressure to be injected against. This same apparatus with injector is sometimes necessary on pumped supplies in case the pump is below the hydraulic gradient; that is, if there is any positive pressure in the pump suction line at the point of application.

In the case of a reservoir with a gate well where there is no water supply under pressure available, we use a direct-feed chlorinator; that is, one where the chlorine gas is fed directly to the water through diffusors. Fig. 3 shows a chlorinator of this type. In this case, the measurement of the rate of flow, instead of being by a volumetric measure as in the case of the solution type equipment previously shown, is indicated by the difference in pressure on the two sides of an orifice, which difference in pressure is measured by a manometer device. In case the point of application is directly into a main, instead of having the run from the control panel to the check valve, the connection is made through a trap tank assembly and the diffusor is tapped into the main through a corporation cock with housing, as shown in the middle of Fig. 3. In those larger capacity plants where pumping equipment is in use and where a water supply of at least 25 lbs. per square inch is available, a further modification with this same control panel is utilized to make what we term our type MDAM with injector machine. An injector panel with check valve is utilized and a lava seated check is inserted in the discharge line from the injector. The chlorine solution is applied through a corporation cock housing a silver tube. This adaptation permits of introduction of chlorine solution against either a vacuum or pressure. The operating pressure required for the injector is dependent on both the amount of chlorine to be handled and the pressure to be injected against. This same equipment without a connection to the main may be used for application to open body of water as a suction well.

A development within the last two years is the use of high-pressure reinforced injectors to permit of introducing chlorine into lines against pressures hitherto too high to permit of applying chlorine.

The equipment which is utilized is essentially that shown as the MDAM with injector, utilizing a water pressure going to the injector with a pressure practically four times that to be injected against. To obtain this increased high pressure, two courses are open and are being utilized, one a triplex pump with pressure relief valve and the other, a hydraulic valve mechanism pump. With this type of equipment there have been installed a number of plants where chlorine solution is being utilized against as high as 75 to 125 lbs. per square inch, and a few where the pressure is as high as 150 lbs. per square inch.

It often is necessary to provide chlorine control equipment with an automatic control so that the machine will automatically vary the rate of flow of chlorine in proportion to the flow of water in the main. In such cases a Venturi tube of proper design is inserted in the main to give suitable differential pressures varying with the rate of flow in the main, and in such cases, instead of having the manual compensator (constant rate of flow device) as shown in Fig. 3, an automatic compensator is furnished in its place and the leads from the Venturi tube are connected to this compensator. The automatic compensator, instead of giving a constant flow of chlorine, varies the chlorine flow in proportion to the flow of water through the Venturi tube.

The varying conditions which present themselves as the engineer studies each water supply to determine the type of chlorinator which will best meet existing conditions have been met in the main by the use of the machines just described.

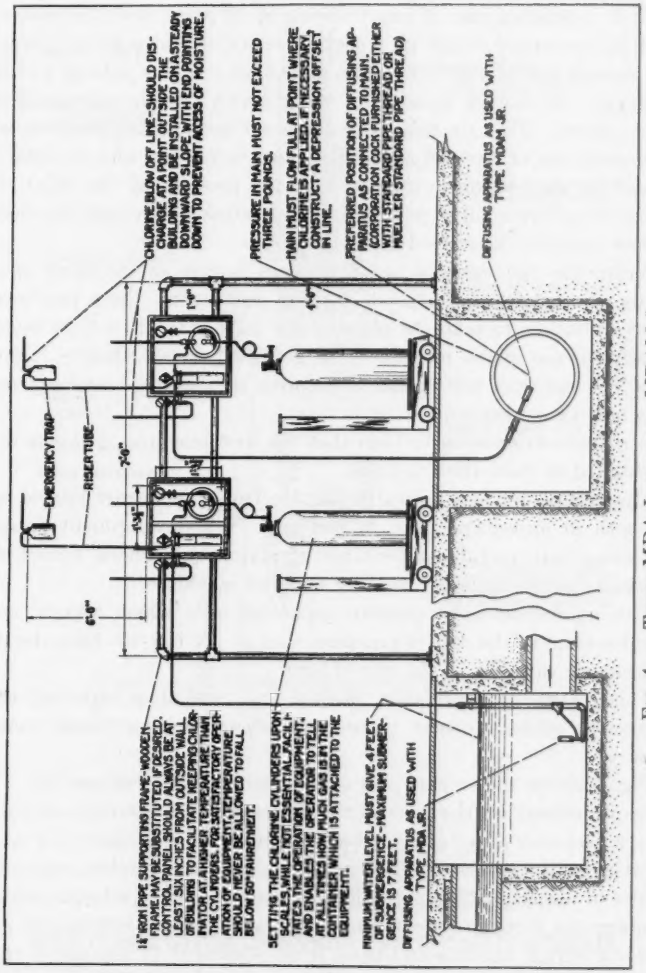

CHLORINE BLOW OFF LINE - SHOULD DISCHARGE AT A POINT OUTSIDE THE BUILDING AND BE INSTALLED ON A STEADY DOWNWARD SLOPE WITH END POINTING DOWN TO PREVENT ACCESS OF MOISTURE.
PRESSURE IN MAIN MUST NOT EXCEED THREE POUNDS.
MAIN MUST FLOW FULL AT POINT WHERE CHLORINE IS APPLIED. IF NECESSARY, CONSTRUCT A DEPRESSION OFFSET IN LINE.
PREFERRED POSITION OF DIFFUSING APPARATUS AS CONNECTED TO MAIN. (CONNECTION CAN BE MADE EITHER WITH STANDARD PIPE THREAD OR WITH MUELLER STANDARD PIPE THREAD)
DIFFUSING APPARATUS AS USED WITH TYPE MDAM JR.
EMERGENCY TRAP
RISER TUBE
2'-6"
1'-4½"
6'-0"
1¼" IRON PIPE SUPPORTING FRAME - WOODEN FRAME MAY BE SUBSTITUTED IF DESIRED. CONTROL PANEL SHOULD ALWAYS BE AT LEAST TWO FEET FROM OUTSIDE WALL OF BUILDING TO FACILITATE REPLACEMENT OF CYLINDERS. FOR SATISFACTORY OPERATION OF EQUIPMENT, TEMPERATURE SHOULD NEVER BE ALLOWED TO FALL BELOW 60° FAHRENHEIT.
SETTING THE CHLORINE CYLINDERS UPON SCALES, WHILE NOT ESSENTIAL, FACILITATES THE OPERATION OF EQUIPMENT, AND ENABLES THE OPERATOR TO TELL AT ALL TIMES HOW MUCH GAS IS IN THE CYLINDER WHICH IS ATTACHED TO THE EQUIPMENT.
MINIMUM WATER LEVEL MUST GIVE 4 FEET OF SUBMERGENCE - MAXIMUM SUBMERGENCE IS 7 FEET.
DIFFUSING APPARATUS AS USED WITH TYPE MDA JR.

FIG. 4. — TYPES MDA JUNIOR AND MDAM JUNIOR.

There developed, however, a demand for a machine feeding the gas directly into a gatehouse well or into a main, of lower capacity than could be obtained with the manometer and orifice type metering device. This demand resulted in the development of the machine shown in Fig. 4.

The machine has incorporated in it the low metering capacity of the

volumetric meter such as was previously furnished only in the solution-type machine of small capacity, and the added adaptability for use where no water supply under pressure is available. The metering in this case is by timing the pulsation of gas as discharged from the volumetric meter through a jar of sulphuric acid. The riser tube and the emergency trap shown are to provide a relief in case of any building up of pressure in the meter jar. In such case pressure builds up, a slight amount of acid is blown out of the meter jar and this is collected in the glass trap. It also acts as a vacuum relief device, preventing water from being drawn into the apparatus when it is shut down. This machine is available for open well application where the submergence of the diffusor is not over seven feet and at least four feet, and for application in mains where the pressure at the point of application is not over three pounds. It has satisfactorily met the demand for a low-capacity direct-feed machine.

Within the last two and one-half years, largely as the result of a call for a more rugged and foolproof type of equipment, there has been developed a machine to take the place of the former solution type machine, especially for use in the medium or larger sized places where a minimum capacity of less than one pound of chlorine per twenty-four hour rate of feed is not demanded.

It may be interesting to note that the first machine of this new type was installed at New Haven, Conn.

This equipment is a radical departure from the former line of equipment both in appearance and in principle. Its development permitted doing away with metal compensator regulators and check valves which had been essential parts of the older types of machines.

This equipment is at present applicable only where manual control or hand setting of the rate is practical, and as yet has not been developed for automatic variation of the rate of feed.

This feature is now being worked out, and it is expected that it will soon be added to meet those cases where such automatic control is necessary.

Fig. 5 shows the control unit of the machine just referred to. One is at once impressed by the general ruggedness of the assembly mounted as it is in a cast-iron pedestal with door and within a vulcanite tray which is supported by the pedestal. The gas is taken from cylinders connected to a header or directly to the chlorine inlet header on the pedestal and enters the equipment through a float-controlled valve. From here on the system is under a partial vacuum and there is no possibility of leakage of chlorine beyond this chlorine inlet valve. Mounted within the bell jar, which sets in the tray in a water seal, is a glass meter tube which has an orifice at its upper end.

A water supply under at least twenty-five pounds pressure is brought to the pedestal and operates to furnish a small water supply to the tray with the main flow passing through a tube connection and thence through

a silver injector which is fitted into the base of the vulcanite tray. When this water supply is operating a vacuum is created at the throat of the

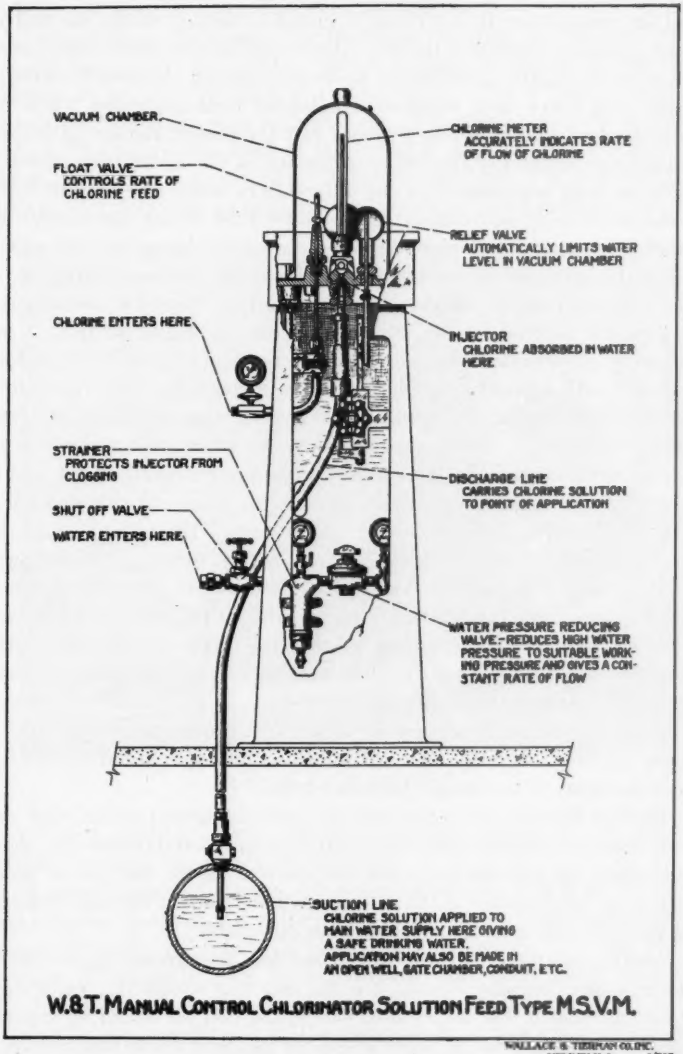

VACUUM CHAMBER
FLOAT VALVE CONTROLS RATE OF CHLORINE FEED
CHLORINE METER ACCURATELY INDICATES RATE OF FLOW OF CHLORINE
RELIEF VALVE AUTOMATICALLY LIMITS WATER LEVEL IN VACUUM CHAMBER
CHLORINE ENTERS HERE
INJECTOR CHLORINE ABSORBED IN WATER HERE
STRAINER PROTECTS INJECTOR FROM CLOGGING
DISCHARGE LINE CARRIES CHLORINE SOLUTION TO POINT OF APPLICATION
SHUT OFF VALVE
WATER ENTERS HERE
WATER PRESSURE REDUCING VALVE.-REDUCES HIGH WATER PRESSURE TO SUITABLE WORKING PRESSURE AND GIVES A CONSTANT RATE OF FLOW
SUCTION LINE CHLORINE SOLUTION APPLIED TO MAIN WATER SUPPLY HERE GIVING A SAFE DRINKING WATER. APPLICATION MAY ALSO BE MADE IN AN OPEN WELL, GATE CHAMBER, CONDUIT, ETC.
W.&T. MANUAL CONTROL CHLORINATOR SOLUTION FEED TYPE M.S.V.M.
WALLACE & TIERNAN CO. INC. NEWARK, N.J.

FIG. 5.

injector, and the interconnections are such that this vacuum pull is transmitted through a silver tube within the meter tube. It is so arranged that when this injector is pulling, water from the tray rises within the glass

meter tube until it overflows at the elevation of the top of the silver tube and passes down the latter to satisfy the vacuum. As this vacuum pull starts to raise the water it also creates a suction through the orifice in the top of the meter tube, thus tending to create a vacuum within the bell jar. As this tendency continues, the ball float operating the needle valve on the gas line raises slightly, a sufficient amount to permit chlorine to enter the bell jar; this valve then becomes a balanced float-controlled valve, adjusting itself to a point where it passes just the proper amount of chlorine to supply the demand of the orifice in the top of the meter tube under the differential head represented by the difference in water level within the jar and the water level as overflowing the silver tube within the meter tube. All that is necessary to change the rate of flow is to change the elevation to which water can rise to establish the differential head operating on the orifice. This is done by means of a ratchet device, rugged in construction, by which the position of the inner meter tube is varied so that it may be set for whatever rate is desired within the range of the meter by observing its position with regard to calibrations on the meter tube. Any one meter is guaranteed accurate within four per cent over a range of four to one (maximum to minimum). The largest capacity for which this machine is furnished is 300 lbs. maximum and 75 lbs. minimum guaranteed feed, and the smallest is 4 lbs. maximum and 1 lb. minimum. These rates of feed are per twenty-four hours. If a greater range is desired than four to one, this can be obtained by the use of two or more meters, by simply removing the bell jar from the water seal, loosening the hard rubber nut which holds the meter in place, and substituting a meter with a different sized orifice replacing the bell jar and resuming operations. There are no gas joints to be broken, as in the case of the older line equipment. In case of a desire for extreme range within one control unit, it is sometimes necessary to furnish an injector or injectors of different design to be changed for the original injector assembly, but within reasonable and usual demands all that is necessary is to change the meter tube.

Another feature which has not yet been mentioned is the relief valve shown in the photograph which automatically opens and breaks the vacuum should there be any tendency for the vacuum in the bell jar to become extreme, as, for instance, if the chlorine cylinder should become exhausted. This makes flooding of the chlorine lines impossible.

Another feature of this machine which has been greatly appreciated by those who have become users of it is the fact that should the water supply fail for any reason, the water level within the bell jar would be depressed by the incoming gas to the point where the gas inlet valve would close and shut off the chlorine feed. This is a feature not true of former types of equipment.

One point which is to be particularly noted in connection with the machine is the complete use of materials which cannot be injured by chlorine gas. All of the parts within the bell jar are either silver, vulcanite,

glass or other noncorrosive material. The discharge line is special composition tubing and the entry of solution into the main is through a silver tube. Further, inasmuch as the machine cannot be flooded back into the gas lines, corrosion of these lines does not take place.

The operator appreciates the fact that the parts of this machine are so easily dismantled and cleaned that it is a simple matter in case of any difficulty to get at every part of the machine. This is done without the necessity of breaking joints with gaskets. The simple removal of the bell jar from the water seal renders the chlorine inlet valve and meter tube readily accessible or removable. With this machine it is possible to feed into a suction line, into an open body of water, or into a pressure line where

FIG. 6. — CHLORO-BOAT.

the pressure existing is not over approximately one-fourth of the water pressure available to operate the machine.

A large number of these machines are now operating in actual service, some for about two years, and they have given extreme satisfaction, and the general favorable comments of plant operators where they are installed are most enthusiastic. Up to the present time there have been about three hundred units of this type of equipment installed.

The feature mentioned in the description of this machine by which the machine shuts down if the water supply is cut off has been utilized to render this machine applicable to use for what is termed as semi-automatic control. At Scarsdale, N. Y., either one or two pumps are in operation automatically, each at a fixed rate according to the pressure existing in the system. By the installation of vanes in the discharge line from each pump,

these vanes operating valves in the water supply lines to the chlorinators, it is possible to have either one or both of the chlorinators in operation.

The general ruggedness of this equipment has rendered it adaptable to a use which may be of interest, and Fig. 6 shows the chloro-boat, utilized at the Potomac Tidal Basin for sterilizing this bathing place. These chlorinators discharge chlorine water through hose lines astern of the boat and the boat circles about the basin sterilizing the water.

The rugged construction of this equipment has rendered it particularly applicable for use in connection with construction work which is being performed upon some of the larger watersheds. Seven of these machines have been installed in connection with contractors' work on the New York Water Supply where tunnels or siphons are being constructed, the work being located on the watersheds of other towns. The Board of Water Supply of New York required that all water pumped from these workings be sterilized before discharging into the sheds. The same practice was carried out in connection with the new water supply for the city of Providence, R. I., and two such machines are installed there for this purpose.

It is now an established fact that this vacuum type solution machine has met the demands for a rugged and practically foolproof chlorine control apparatus and it is being generally installed where applicable.

We have endeavored to present to you some of the most recent developments in chlorine control apparatus for use on water supplies and to show you the advances which have been made in the matter of control equipment, both in the matter of machines to meet varying conditions and in the way of improvement with the end in view to develop as rugged equipment as possible, which is as near foolproof as it can be made.

STANDARDIZATION OF FIRE HOSE THREADS.

BY MAJOR J. HASTINGS HOWLAND.*

[*Read December 11, 1923.*]

The invitation extended by your President to present at this time a paper before this great service Association was keenly appreciated. For some two years or more the National Board of Fire Underwriters has been seeking an opportunity to emphasize the urgent need of greater initiative on the part of water works men and organizations towards the country-wide Standardization of Fire Hose Threads.

The speaker has had this movement in charge since it was first taken over by the National Board some 5 to 6 years ago, and is in a position to know, after 35 years' experience in water works studies, that it spells — ECONOMY — EFFICIENCY and CONSERVATION, to every system of public and private water supply in the country.

For more than half a century the distinct advantages to be derived from the Standardization of Fire Hose Threads have been generally recognized. But up to very recent years it was with this subject, a good deal as Mark Twain once said about the weather, " that there was a whole lot of people talking about it, but none of them were doing anything about it." It has, however, taken on a new lease of life, with the result that the fire chiefs and general public, as never before, are becoming aroused with the conviction that " Misfits in Fire Hose Threads " must no longer be tolerated. Should the water works men, as guardians of the greatest of all fire-fighting facilities, show less initiative than our fire chiefs and other individuals and organizations that have recently become actively engaged in applying the remedy?

When business is good fires decrease, and when business is bad they increase, but when the fire hose threads in any city or town cannot be connected up with those of its neighbors, it doesn't matter whether business is good, bad, or indifferent, waste from fire is sooner or later bound to follow.

Unmateable threads have contributed to the spread of scores of our most destructive fires. Boston had its experience 51 years ago. There was a time when that city could not connect up with the threads in Cambridge, directly across the Charles River. New Bedford and Fall River have extended their built-up areas so that they have almost grown together; their existing hose threads are not interchangeable one with the other. Twenty years ago on the 7th of next February, leaping, gale-driven flames threatened the city of Baltimore with extinction. Not one of the big,

* Engineer, National Board of Fire Underwriters.

strong companies responding from the Washington, Philadelphia, and New York fire departments could connect their hose to the Baltimore hydrants; uniform threads would undoubtedly have saved quite a few of the *fifty millions* in property values that were destroyed in that conflagration. There are scores of instances, three in the State of Maryland within the past two years, where unmateable coupling threads were the outstanding cause of the huge and senseless losses that were experienced.

How many of you ever stopped to think that human lives and property values in your respective municipalities are more dependent upon those threaded hydrant hose outlets than upon any other fire-fighting facility in existence? Can we water works men, fire chiefs, and other officials directly in charge of these public utilities escape our full share of the responsibility for continuing as in the past to permit different sections of the country, various cities and towns in every state and in a number of instances a single municipality, to attach widely different and frequently non-interchangeable threads to the indispensable fire hose and hydrant outlets?

What is more to the point, are we going to be allowed to let this awful toll continue to be exacted, through our purblind self-complacency? While the development of the existing thread situation was inexcusable from the very start, the effective application of a very simple and inexpensive remedy has already gained irresistible headway, with every indication of its steadily attaining increased speed.

Let me review briefly what has been accomplished since the National Board first took hold of this movement nearly five years ago.

Complete sets of tools for converting the great majority of the existing threads to the universally adopted National Standard, the lack of which had been an outstanding factor in retarding progress throughout this long term of years, were first made available and tried out in May, 1919, in Hunterdon County, N. J. Two months later the threads in three municipalities in Indiana were completely standardized.

The results thus obtained were so encouraging that organized movements for state-wide standardization were then and there suggested. Pamphlets issued and widely distributed late in 1919 and in 1922, helped to arouse further interest.

Michigan and New Jersey in 1920 took the lead; followed by Indiana, Minnesota, Ohio, and Rhode Island in 1921. As a result we now have over one-third of our states in which this movement is organized and actually under way. Since the spring of 1920 close to 800 of our municipalities have had their fire hose threads completely standardized, about 250 of which were finished up during the present calendar year. There are approximately 8 000 protected cities and towns in the entire country; it is therefore evident that we have made some considerable progress.

Many public and private organizations have rendered valuable assistance in advancing this movement. The Hackensack Water Company

voluntarily standardized the threads on all of their hydrant nipples and caps and much of the fire station equipment in the 50 towns which it serves. Many of our sectional and state associations of fire chiefs have appointed working committees that are getting good results. The National Screw Thread Commission has adopted definite thread limit dimensions for the previously adopted national standards. These with specifications for the general adoption of proper gages for testing the finished product, both by the manufacturer and the user, have been embodied in pamphlets prepared by the American Society of Mechanical Engineers, which are now before the American Engineering Standards Committee for approval. The Brass Hose Fittings Manufacturers' Association, which represents three-fourths of the hose coupling output in the country, has made provisions for checking the finished product by means of complete sets of thread limit gages, in all future cases, where national standard threads are specified. Considerably over a year ago we solicited, but without effect, an invitation to give a talk and demonstration before the Water Works Manufacturers' Association with a view to getting similar action, thereby relieving its membership of unsatisfactory jobs, involving replacements, alterations or repairs of threaded hydrant parts for fire hose connections. The National Fire Waste Council under the leadership of the Chamber of Commerce of the United States has embodied the Standardization of Fire Hose Threads as a feature of the Inter-Chamber Fire Waste Contest, in which many of the 1 400 local chambers of commerce and trade organizations are participating. In February of this year the Oregon State Legislature passed a bill making it mandatory for all state and municipal fire hose fittings to be equipped with national standard threads. Other states have sent for copies of that bill. Under the direction of the state fire marshal, 16 of their municipalities had, to November 1 this year, been completely standardized. Some of our railroads, through the efforts of a Committee of the Railway Fire Protection Association, have purchased tools for the purpose of converting the threads on their respective properties, and influencing some of the towns along their lines to do likewise.

We are safe in stating that no similar movement heretofore started has met with stronger endorsement, greater coördination of effort or more generous public support. Yet we are far short of the desired goal, as indicated by the fact that approximately three-fourths of our 8 000 protected cities and towns still have non-standard threads.

The New England and the American Water Works Associations were among the very first to officially endorse and approve the specifications, first recommended by the committee of the National Fire Protection Association in 1905, for the national standard threads. In all 29 of our leading engineering societies, water works, and fire chiefs' associations, and national, state, and private organizations have taken similar action. So that this Association helped to get this standard thread so strongly supported and adopted that it cannot now be supplanted.

Can this Association, however, afford to be content with passive, rather than initiatory, action in so important a movement in public safety and in which the utilities, placed under your supervision, play so large a part?

DISCUSSION.

MR. THOMAS E. LALLY.* Mr. President, I would like to ask Mr. Howland what is the nature of the metal upon which they have changed the threads? Was it brass or composition?

MR. HOWLAND. I am glad to answer that question. At the last meeting of the Brass Hose Fittings Manufacturers' Association they adopted specifications for the composition of hose couplings, which should include hydrant nipples, and they are adhering to and have adopted the Underwriters' Laboratories' specifications for hose couplings, requiring 83 per cent of copper, about 5 per cent of tin and small amounts of zinc and lead. There has been a very wide tolerance practiced in the manufacture of composition. We found in Connecticut not long ago, in expanding a $2^{31}/_{32}$-in. up to the National Standard diameter, we cracked perhaps 25 per cent of the hydrant nipples in the town of Manchester; upon examining those nipples we found they contained a large percentage of zinc, very white, sandy and friable; not only poor composition but extremely thin, so that they would not stand expanding. The town looked at it from a broad viewpoint and ordered National Standard nipples put in. They are entirely different looking nipples from what they had originally. But I will say that in all these 800 cities and towns that we have standardized we have not cracked 1 per cent of the nipples, so do not gather from the experience of this small town in Connecticut that 25 per cent of the existing undersized nipples are going to be fractured. It is misleading. The composition of hydrant nipples is a matter that should be taken up, I think, by water works associations and fire chiefs' organizations throughout the country and made a national standard, because the manufacturers, and I suppose some of the jobbers and unscrupulous fellows, are putting in all kinds of metal. I heard in New York of one jobber or small manufacturer, a shopman, who was making fire-hose couplings and selling them for $1.60 a set, where the recognized manufacturers, like the Acorn Brass and some of the others cannot make the National Standard composition and weight for less than $3.50 or $3.60. You can see what is going on.

MR. JOHN C. CHASE. I would like to inquire how large cities have adopted this new standard?

MR. HOWLAND. Flint, Fort Wayne, Evansville, Memphis, Saginaw, Chattanooga, and other cities, some of 100 000 and over, have been completely standardized; and in connection with the recent convention of the International Association of Fire Engineers we had a meeting of the eastern

* Assistant Engineer, Water Service, Department of Public Works, Boston, Mass.

metropolitan city fire chiefs: from Boston, both Chief Taber and Commissioner Glynn; the Philadelphia fire chief, the Washington fire chief, the Richmond chief and commissioner and from Baltimore one of the Board of Fire Commissioners and the fire chief. We talked over this proposition and I thought we did well; we got the city of Boston and the city of Washington to authorize us to say that standardization was to be at once taken hold of in those two cities. Commissioner Glynn and Chief Taber issued an order that nothing but National Standard threads are to be purchased from now on. Of course they have got an easy problem. They have the Roxbury thread here, which is mighty close to the National Standard. It is supposed to be the same diameter, with only a half thread difference in an inch of travel; all they have got to do with the Roxbury thread is to bring it right to diameter so as to interchange with the National Standard. I know from the processes of manufacture heretofore used that the product is not uniform; they are going to have some of those Roxburys that are going to clash with the National Standard thread. It means they have got to spin over with their tools some of the existing threads to get good fits. Boston and Washington have authorized it and we have a large number of cities of over 50 000 that have completely standardized their fittings within the past three years.

MR. E. D. ELDREDGE.* Mr. President, I would like to ask if the speaker will please repeat the dimensions of the standard thread — whether it is V thread or United States thread, and the pitch or the threads per inch?

MR. HOWLAND. It is known as the National form of thread, 60 degrees V thread with the specified flats in the valley and peak, $3\frac{1}{16}$ in. outside diameter of the male and $7\frac{1}{2}$ threads per inch.

MR. LALLY. Mr. President, I would like to ask if anything was done on the standardization of the steamer connections.

MR. HOWLAND. This Association adopted the National Standard thread, which included everything from $2\frac{1}{2}$ in. up to and including $4\frac{1}{2}$ in.; but we have not yet found any practical means of standardizing the $4\frac{1}{2}$-in. fittings, because the variations are too wide for any one set of tools to handle. The $4\frac{1}{2}$-in. is the only recognized steamer suction diameter, if that is what you want.

MR. LALLY. That is the point I was getting at.

MR. HOWLAND. It is the only National Standard thread for steamer connections.

MR. LALLY. The reason I asked is because there has been some talk lately about cutting down the size of steamer connections; that there was less strain on the hydrant due to the discharge through small connections than through the large ones. Having had some experience in hydrants and in testing hydrants, I was curious to know whether the National Board had made any recommendations on that point.

* Superintendent, Onset (Mass.) Water Company.

MR. HOWLAND. I think that inasmuch as the automobile pumping engines are getting larger and larger, some of them being made over one thousand gallons, I do not think the National Board would want to go on record as advocating anything less than that the National Standard, $4\frac{1}{2}$-in. suction for steamer connections.

CAST-IRON SERVICE PIPE.

By J. R. MC WANE.*

[*Read December 11, 1923.*]

Some years ago a committee of your Association made a study of service-pipe problems and submitted a report in which they listed six essential requirements for a satisfactory service pipe. Now, with the vast increases in street paving following the rapid growth of urban population in this country, has come a fresh realization of the enormous wastes of digging up these pavements to repair and replace services. When added to the labor bill, the rust bill already levied, the total price of impermanent pipe for services has appeared prohibitive. So it is with a view of discussing the application of your committee's recommendations to small cast-iron pipe for services that the following remarks are addressed.

These recommendations, as you may recall, were:

1. That the service pipe should not affect the water passing through it in such a manner as to make it injurious to the health of those using the water.
2. That it should not have a deleterious effect on the taste, or odor, of the water, even though not injurious to the health.
3. That it should have a sufficient capacity to give adequate service at all times.
4. That it should be strong and durable.
5. That it should be easily laid.
6. That it should be inexpensive.

Now all of this sounds at first blush as though it were a pretty accurate description of a cast-iron service. But it has not always been, and there are several interesting and important developments in sizes, methods of casting, and of making joints that go far toward putting a new and brighter face on the matter of using cast-iron pipe for services.

Taking up the committee's requirements in order, there is little new to be said concerning cast-iron pipe under the first two heads. The same kind of pipe and coating as the mains themselves makes unnecessary any separate consideration of mains and services when both are cast iron. It guarantees that there will be no harmful effects as to taste, odor, or health.

Cast-iron service pipe would remove all fear of metal poisoning, for it produces no compounds injurious to the health of those using the water. Even if the standard pipe coating were to be damaged, there would merely form a protective coating of rust on the inner surface of the pipe which would become a part of the pipe itself. It would not come off in the water.

* President, McWane Cast-Iron Pipe Company.

The third requirement has heretofore been more often the cause of complaint from quite the opposite standpoint: not that cast-iron pipe was too *small*, but that it was too *large*. And, with the smallest cast-iron pipe in common use being 3-in., there were numerous cases where cast-iron pipe was too large. Feeding a five-room residence or apartment through a 3-in. pipe, for example, was poor practice from more viewpoints than one.

Happily, this condition has been met. At our foundries in Birmingham we are now successfully making cast-iron pipe as small as 1¼ and 2-in. This not only insures sufficient capacity without wasteful over-capacity, but an interesting side light has been developed by V. Bernard Siems, Engineer for the City of Baltimore Water Department. Studies and tests made by him with a number of 1¼-in. cast-iron services there have revealed a great reduction in friction loss. His comparative charts covering this feature are most interesting.

Particularly does cast-iron pipe shine under the fourth head, that the service pipe shall be strong and durable. There is also a strong " home flavor " to this portion of the subject. New England has undoubtedly made more real study of the causes of and cures for service-pipe troubles than any other part of the country. In consequence, it is here too that perhaps the largest number of cast-iron services are already in use. In spite of some very real objections to them — which we believe we have now overcome — many cast-iron services are in satisfactory use in this part of the country to-day.

As to the durability of the material from which cast-iron pipe is made, it is felt that no further briefs need be advanced for cast-iron. I wish I could have brought with me some portions of a 2-in. cast-iron service which were dug up in Hagerstown, Md., this year and sent to us. This service was laid in 1881, and when unearthed, after 42 years' faithful service, was to all appearances as sound as the day it was laid.

Records here and abroad still do not extend far enough to indicate the real life of cast-iron pipe. We can only say that *if* and *when* the cast-iron pipe in some of the oldest lines in France, now completing its third century of continuous service, finally wears out, then we shall know what the maximum life of cast-iron pipe is. This long durability native to all cast-iron pipe is, of course, one of the strongest reasons for considering it for services.

As for strength, we are now producing by the McWane process of casting in green sand a far tougher and stronger pipe than it has heretofore been possible to get. It is a very close-grained iron of great tensile strength. It reduces breakage to a minimum — an important point with small pipe.

Another feature of the dependability of the service as a whole when laid with this pipe is the flexibility and automatic joint tightening introduced by the McWane precalked bell and spigot type of joint. It will be described in detail a little later.

It has been thoroughly demonstrated in the experience of past years that a screw-joint cast-iron pipe in the smaller sizes is not satisfactory. The weakest point in such a line is at the base of the thread. It is a well-known fact that the breaking of the skin on cast iron weakens it out of proportion to the amount of iron removed. In other words, the strength of a casting is largely in the skin, which is close-grained and tough, due to the change of the nature of the carbon as a result of more sudden cooling next to the sand. Most of the trouble with cast-iron service pipes which were used before the advent of steel was breakage at the threads, on account of weakening, as stated above, and on account of rigidity of the lines. In the development of McWane pipe for services this has been kept constantly in mind. Outside threads are used only when absolutely necessary to connect with existing lines or fittings. The lead gooseneck takes the strain off the thread at the connection with the main. Moreover, all ends of pipe or fittings to be threaded are restricted in their internal diameter to compensate somewhat for the weakening at the thread. This restriction of the diameter at one or two points does not materially affect the flow, as evidenced by the Venturi meter. Both male and female fittings are furnished for 1¼-in. pipe and the preferred practice is to use the female fittings for connection both at the main and the curb, which entirely obviates any danger from an outside thread. However, this danger, as stated above, is reduced to a minimum by the strengthening of the section where the thread is to be cut. While McWane pipe may be threaded at any point in an emergency, it is not recommended by the manufacturer as good practice. Threads should be cut only on ends that are reinforced for the purpose.

It is the substitution of an improved bell and spigot joint for the threaded couplings of former years that has perhaps done most for the renewed consideration of cast-iron services. Liability to breakage at the joints under earth settlement and traffic has thus been practically eliminated. A service of the new type is capable of resisting practically every strain, settlement, or vibration that may be thrown upon it.

Cast iron's higher resistance to electrolysis should also be mentioned as another phase of its durability.

The committee's fifth provision, that the service should be easily laid, is of prime importance. A slow, cumbersome, expensive laying process would go far to defeat the economy and popularity of any type of service. Here, again, has been one of the chief obstacles in the way of more extended use of cast-iron pipe in small sizes. With the positiveness and flexibility of the bell and spigot joint felt to be a necessity for best results with cast-iron pipe, there has been much to say against carrying about a lead pot, yarning, digging bell holes, and all the rest of it, in order to lay small pipe. All this paraphernalia and labor did not fit in with the average practical man's ideas of ease, speed and economy of laying services — and rightly so. Even if the field-made bell and spigot joint were as cheap and reliable

as the factory-made, the consideration of obstruction to traffic would still entitle the latter to preference.

With pipe so equipped, it is only necessary to assemble the lengths, enter spigot ends in bells, cold calk the top halves of the lead joints, and "touch up" the bottoms with offset tools (without bell holes). About the same speed is made as with steel pipe laying. If desired, several lengths can be calked together on the bank and placed in the trench as one. As high as 16 lengths have been so laid as one piece with our 2-in. pipe.

The final provision, that the service should be inexpensive, is bound up in several of the former ones. Low cost of the pipe, of joint materials, and the minimum of time and labor in laying are requisites of a low initial cost. Thereafter long life, with freedom from repairs and replacements, is also necessary to complete the only true economy — a low ultimate cost. Cast-iron pipe combines these qualities, being moderate in first cost and lowest of all in final cost. Weighing advantages and economies against any possible disadvantages, cast-iron services have the balance decidedly in their favor.

McWane service pipe is cast in 5-ft. lengths with the possibility of this being increased to 6-ft. lengths. This facilitates handling and affords enough joints to give the flexibility the line needs.

As it comes from the foundry the bell has a double lead groove in its inner surface. These grooves doubly lock the joint materials in place in the bell.

In the joint-making department these pipes are stood vertically in racks, bells upward. Into each bell goes a mandrel representing the next length of pipe. Around this mandrel, in the bottom of each bell, is placed a layer of braided hemp, absorbent of water and designed to swell and close very tightly the space between the two lengths of pipe when a joint is made. On top of this ring of hemp goes a circle of metal wedges, embedded in their upper ends in a ring of lead. These wedges are the self-tightening device mentioned. Under any movement of the pipe they slide a fraction of an inch forward, compressing one of the layers of hemp and automatically tightening the joint and preventing leaks.

Next come two more layers of braided hemp calked in place. Then melted lead is poured into the joint, completely filling it flush with the top of the bell. Finally the half of the joint circumference that is to be placed downward in the trench is precalked with a straight tool, following which the mandrel is removed and the wooden blocks are inserted that protect the joints while in transit.

So you see you have the old reliable bell and spigot joint. The main difference is that the joint is made at the factory instead of in the trench. Instead of *making* it, you merely *complete* it by top calking it and touching up the bottoms. This cold calking of the top halves of the lead joints drives the pipe downward to a firm seat at the bottom, and the "touching up" finishes the operation. You have a uniform, positive, flexible, yet

rigid, self-tightening joint. You have got rid of your lead buying, melting, and pouring; your yarning, your bell-hole digging, and half of your calking. And you have a factory-made joint instead of a "homemade" one, at a saving of time, money, labor, and leaks.

The male and female pieces referred to above, which are used for getting away from and getting into corporation and curb cocks, are furnished threaded for whatever size is wanted, ½-in. to 1¼-in., and in either bell and thread or spigot and thread.

A slightly different application of the same principle is used with 2-in. and 3-in. pipe by means of double hub lengths and of double-threaded lengths, which latter may be cut and fittings inserted where desired. The double hub pieces permit resumption of the hub leading in laying the line.

No attempt is made in this paper to go into all the detailed methods of connecting up cast-iron services, it being felt that a general description of the requirements, of the pipe that has been produced to meet them, and of the method of changing from bell and spigot to threaded connections is sufficient. With the principles clear the rest is detail.

Recent happenings indicate that the various cities of the country are becoming increasingly awake to the desirability of more cast-iron services. Washington, D. C., has lately moved along this line in amending its plumbing ordinances so that only lead, drawn-brass, or cast-iron services may be laid under its paved streets. Of course cast-iron has preference under these regulations from the standpoint of initial cost in *competition* with these other materials.

The Lake Forest, Ill., Water Company has lately notified its consumers that:

> "It has seemed advisable to adopt a metal that will not rust or deteriorate under ground. To this end it has been decided that extra heavy lead pipe or CAST-IRON WATER PIPE will be uniformly required from the street main to the water meter on all new installations for private service lines.
>
> "Entire replacements of leaky services will be considered as new service and will be governed by the same regulations."

This is only typical of what is taking place at many points. And with the removal of the final limitations to the extensive use of small cast-iron pipe, as just described, it is apparent that the day of the economical and enduring cast-iron service is at hand.

FURTHER STUDIES OF WATER PURIFICATION BY FILTERS CHARGED WITH ALUMINUM OR FERRIC HYDROXIDE.

BY H. W. CLARK.*

[*Read February 12, 1924.*]

At the annual convention of this Association held at New Bedford in 1922 I read a paper in regard to the treatment or purification of water by slow sand filters loaded or charged with aluminum or ferric hydroxide. That paper reviewed the results obtained at the Lawrence Experiment Station during five years' trial with small filters of this new Lawrence method. Since that convention a further large amount of interesting information concerning this method has been accumulated by us at Lawrence and elsewhere and I propose to give some of that information at least in this paper.

To those who have not read the previous paper, allow me to state that these loaded filters are primarily for the removal of color and organic matter from water and such results as we have obtained seem to show quite conclusively that this removal of color and organic matter, especially with the soft New England waters, can be obtained rather more cheaply than by mechanical filtration and produce an effluent no more corrosive than the water undergoing treatment.

Description of Process.

Briefly, the process as developed by us consisted at first in loading sand filters with either aluminum or ferric hydroxide by the application of alternate doses of soda ash in solution and either aluminum or ferric sulphate in solution, aluminum or ferric hydroxide being precipitated on the sand grains throughout the entire depth of the filter. When constructed of sand of the grade quite commonly used in slow sand filtration, that is, with an effective size of from 0.25 to 0.30 mm. and operated at a rate of 5 000 000 or 6 000 000 gal. per acre daily, they remove for periods of from 5 to 8 weeks from 60 to 75 per cent of the coloring matter of the applied water. When constructed of finer sand, that is, sand with an effective size of from 0.10 to 0.15 mm., they will often remove more than 90 per cent of this coloring matter for periods of 3 or 4 months. At the end of each run or period of successful color removal it is necessary to remove the coloring matter accumulated within the filter by the application of a solution of caustic soda, this solution generally being passed through the filter several times.

*Chief Chemist, Massachusetts Department of Public Health.

Method of Loading.

From the beginning, two methods of loading or charging the filters were used. The first method and the one absolutely successful at Lawrence was the application of alternate doses of solutions of aluminum or ferric sulphate and solutions of soda ash. Further experience demonstrated that this method was not successful with larger filters, as it was impossible to prevent unequal precipitation of the hydroxide in the filters, hence stratified layers were formed. These stratified layers clogged the filters, caused great loss of head, which destroyed their rate efficiency, and shortened the runs between caustic treatments. The filters can be evenly charged, however; that is, the hydroxide precipitated equally throughout the entire depth of filter by using finely powdered magnesite as the alkali and mixing it thoroughly with the sand when the filter is constructed. This mixing can be done with a concrete mixer or other apparatus. Recent experiments have shown that with care, soda ash can be successfully used in the powdered form but the use of magnesite is preferable. When loading with powdered soda ash both this body and powdered aluminum sulphate are mixed thoroughly with the sand during filter construction and the filter slowly flooded from below.

Permanency or Removal of Aluminum Hydroxide.

As the Lawrence filters loaded with aluminum hydroxide gave as satisfactory color removal results at the end of five years as when first put in operation, and as upon examination they were found to contain the theoretical amount of hydroxide, that is, the amount precipitated within them when constructed, the statement was made in the former paper that this hydroxide was permanent within the filter. This is probably true up to a certain point, but as discovered by further work its permanence in the filter depends upon the strength of the caustic solution applied in removing stored color. When regenerating the Lawrence filters we seldom used more than five tons of caustic per acre of filter surface and this caustic has always been applied in comparatively weak solutions. In work at Providence, R. I., however, where some investigations of this process were carried on by the Water Supply Board of that city (Frank E. Winsor, Chief Engineer), under the direction of the writer, attempts at regeneration with strong solutions of caustic resulted in great removal of aluminum hydroxide. These stronger solutions were used, generally speaking, in order to obviate passing the caustic through the filter several times; that is, to do away with pumping over. This pumping over, however, apparently costs but little and hence the use of strong solutions is not generally necessary. The Providence results, however, caused us to make extensive studies on this point of hydroxide removal and a thorough examination of the Lawrence filters to determine their condition. As a result of this it was found that the experiment station filters put into operation in 1917 and 1918 and which

had never been treated with more than five or ten tons per acre of caustic soda, still contained their initial amount of aluminum hydroxide. Frequent examinations of the caustic solutions after passage through these filters made during the years of operation showed also that the amount of alumina present in the used caustic, that is, removed from the filters during regeneration, was no greater, generally speaking, than the amount normally present in the river water applied to the filters and perhaps deposited in the filters between caustic treatments.

Another factor to be taken into consideration was the fact that the total amount of hydroxide removed by regeneration in five years of operation of the Lawrence filters was but a small fraction of the amount of alumina primarily present in the sand of which the filters were constructed before any hydroxide was precipitated within them; that is, before the filters were loaded or charged. The sand used in filter construction both at Lawrence and Providence contained naturally from 10 to 12 per cent by weight of alumina. Application to certain of the Lawrence filters of strong caustic solutions — 30 or 40 tons per acre — resulted in the same great removal of aluminum hydroxide as at Providence.

Ferric Hydroxide.

Notwithstanding the satisfactory Lawrence results, that is, that the Lawrence filters still contain the primary amount of hydroxide precipitated in them when they were started, the fact that overdoses of caustic will remove aluminum hydroxide has caused us to consider more in detail the use of ferric sulphate instead of aluminum sulphate as ferric hydroxide when precipitated in a filter is permanent whatever the strength of caustic used in regeneration. In fact, two of our early Lawrence filters were charged with ferric hydroxide in May, 1917, and September, 1918, — six and one-half and five years ago, respectively, — and both have given as good color removal as any of the Lawrence filters charged with aluminum hydroxide. The reason that the use of this ferric hydroxide had not been more dwelt upon in the Lawrence work was that ferric sulphate apparently had to be used if ferric hydroxide was to be precipitated in the filter and this ferric sulphate was not, we believed, a commercial product low enough in price to warrant its use in this way.

We have overcome this obstacle, however, in two ways: — (1) We find that a ferric sulphate solution can be obtained from certain chemical companies, this solution being known in the chemical and dyeing trade as nitrate of iron. This term is a misnomer, however, used only for trade purposes and apparently because nitric acid is used as an oxidizing acid in the production of this body. This nitrate of iron, so-called, is really a thick, heavy solution of ferric sulphate. It has a specific gravity of about 1.4 and contains 4.6 lbs. of ferric sulphate per gallon. It can be obtained at such a price that the sulphate present costs only about 2.6 cents a pound,

and as this ferric sulphate is in solution it is 100 per cent available while crude aluminum sulphate as purchased is only 55 per cent sulphate; that is to say, at the figure given, ferric hydroxide can be precipitated in the filters at a somewhat less cost per ton than aluminum hydroxide. This fact makes the use of ferric hydroxide entirely reasonable and practical in filters of this kind, and at the present time we have in operation at Lawrence two sand filters containing approximately 30 tons per acre of ferric hydroxide precipitated from this so-called nitrate of iron solution, and each of these filters is doing good color removal work. This ferric sulphate

TABLE NO. 1.

ALUMINA CALCULATED TO POUNDS PER ACRE FOUND IN WASHINGS FROM FILTERS TREATED WITH CAUSTIC SODA.

Filter No.	Size of Sand. (Millimeter.)	Date of Treatment 1923.	Caustic Soda Used. (Tons.)	Alumina. (Pounds.)
488	.25	Feb. 27	10	148*
		Apr. 9	10	127*
		May 8	10	242*
494	.25	Feb. 27	10	77
		Apr. 9	10	188
		May 8	10	142
		June 25	10	0
512	.25	Feb. 27	5	77
		Apr. 5	5	233
		May 8	5	192
		June 27	5	98
514	.25	Feb. 27	5	127
		May 8	5	133
515	.25	Feb. 27	10	156
		May 8	10	1 232
		June 28	45	15 900
527	.25	Apr. 9	5	273
		May 8	5	83

* Loaded with ferric hydroxide.

solution as now obtained is rather acid. This can be overcome, of course, by using an excess of alkali in filter charging and no doubt a less acid product would be obtained if there was a demand for it. (2) Besides the use of the ferric sulphate solution we have recently discovered a second way of loading a filter with ferric hydroxide and that is by the use of copperas or ferrous sulphate. In other words, we have found that if a sand filter is loaded with copperas and magnesite and run intermittently for a short time, the air entering the filter and the dissolved oxygen in the water applied will oxidize the ferrous hydroxide formed to the ferric state. This apparently simple method did not occur to us until quite recently but preliminary results are encouraging.

TABLE NO. 2.

Filter No.	Filter Started.	Depth of Filter. (Feet.)	Effective Size of Sand. (Millimeter.)	Tons of Aluminum or Ferric Sulphate per Acre.	Tons of Aluminum or Ferric Hydroxide per Acre.	Times Filter was Regenerated Since Start.	Average Number of Days Between Treatments.
488	May 14, 1917	4	.25	64.5*	34.5	39	51
494	June 7, 1918	4	.25	80.5	20.2	35	46
496	Sept. 19, 1918	4	.25	27.0*	14.4	10	128
512	Feb. 2, 1921	4.5	.23	75.0	18.8	16	53
514	Feb. 2, 1921	4.5	.23	150.0	37.6	10	88
515	Feb. 2, 1921	4.5	.23	150.0	37.6	10	85
523	May 22, 1922	4.5	.12	170.0	42.7	4	112
525	June 8, 1922	4.5	.12	150.0	37.6	3	122
527	Feb. 16, 1923	2	.25	125.0	31.4	2	82
528	May 2, 1923	4	.23	150.0	37.6	1	142
529	May 2, 1923	3	.23	112.5	28.2	1	142
530	May 2, 1923	2	.23	75.0	18.8	2	85
532	May 18, 1923	2	.23	150.0	37.6	1	128
533	May 18, 1923	3	.23	150.0	37.6	1	128
534	Sept. 6, 1923	4.5	.23	73.5*	39.3	—	—

* Ferric sulphate.

TABLE NO. 3.

Filter No.	Filter Started.	Depth of Filter. (Feet.)	Tons of Aluminum or Ferric Hydroxide per Acre.	Average Color of Applied Water and Effluent.	Average.		pH.
					Alkalinity.	CO_2.	
Applied Water..	—	—	—	41	1.1	0.4	6.2
488	May 14, 1917	4	34.5*	15	1.5	0.2	6.4
494	Jan. 7, 1918	4	20.2	17	1.2	0.2	6.6
496	Sept. 19, 1918	4	14.4*	10	1.4	—	6.6
512	Feb. 2, 1921	4.5	18.8	16	1.3	0.2	6.6
514	Feb. 2, 1921	4.5	37.6	15	1.3	0.2	6.5
515	Feb. 2, 1921	4.5	37.6	12	1.3	0.2	6.8
523	May 22, 1922	4.5	42.7	8	1.2	0.3	6.5
525	June 8, 1922	4.5	37.6	7	1.3	0.4	6.4
527	Feb. 16, 1923	2	31.4	7	1.4	0.2	6.6
528	May 2, 1923	4	37.6	3	1.1	0.4	6.3
529	May 2, 1923	3	28.2	3	0.9	0.4	6.1
530	May 2, 1923	2	18.8	10	1.9	0.3	6.5
532	May 18, 1923	2	37.6	4	1.4	0.5	6.4
533	May 18, 1923	3	37.6	4	1.3	0.5	6.2
534	Sept. 6, 1923	4.5	39.3*	3	0.9	—	6.0

* Ferric hydroxide.

By this process a slight excess of alkali is necessary to cause the chemical reaction to occur and this excess must be taken up by the applications of small amounts of ferric sulphate. About 90 per cent of the hydroxide loading is accomplished with the ferrous salt and the remainder with the ferric.

Color Removal by Experimental Filters.

At the present time we have fifteen filters in operation at Lawrence as shown by Table 2, all operating at about 5 000 000 gal. per acre daily. Certain of these filters contain from 19 to 37½ tons of aluminum hydroxide per acre and five of them loaded with the use of ferric sulphate contain from 14½ to 39 tons of ferric hydroxide per acre. The table shows that we precipitate in the filter from a given weight of ferric sulphate practically twice as much hydroxide as from the same weight of aluminum sulphate. The table also shows the depth of the filters, the date when they were put into operation, the number of times each filter has been treated for the removal of stored color, the average number of days or length of run between these treatments. This average as will be noted has varied from 51 to 142 days.

Table 3 presents some of the figures of Table 2, together with additional data; that is, it shows the tons of hydroxide per acre in each filter and the depth, as shown on the previous table. Besides these records, however, the average color of the applied water and effluents from the filters are shown, the average alkalinity of the applied water and effluents, and the hydrogen concentration factor of these waters.

It will be noticed that the color removal results vary to some extent, but that the filter giving the smallest average removal, namely, No. 494, shows a reduction of 60 per cent while many of the filters show average color removals of 80, 85 and even more than 90 per cent through a long period of operation. The hydrogen ion concentration range averaged 6.2 for the applied water and from 6 to 6.8 for the effluents. Generally speaking, we have found that when the average pH is from 6 to 6.3 the best color removal is obtained.

Efficiency Lessened by Alkali but Increased by Acid Treatment.

There is one peculiar thing in the operation of these filters and the hydrogen ion concentration factor that has been brought out during the year. Apparently, as I have stated, the best results are obtained when the pH of the effluents is about 6 to 6.3 or slightly more than the hydrogen ion concentration or pH of the applied water. This slight increase in the effluent is due entirely to the residual alkalinity retained by the hydroxides in the filters in the form of sodium bicarbonate as a result of the treatment with caustic soda solutions; that is, some of this caustic soda is converted to bicarbonate by the free carbon dioxide in the water. Now if you increase

the pH factor by adding a little more sodium carbonate, say, enough to increase the pH of a filter effluent from 6.2 to 6.7, the filter immediately begins to produce poor color removal results. Trying this with Filter No. 530, for instance, increased the color of the effluent in a day or two from 6 to 16. This led us to try an opposite method of treatment of the filters; that is, the addition of a slight amount of acid to the applied water immediately after regeneration with caustic. The results are shown in Table 4 and indicate clearly the rapid effect on the color removal efficiency of the filter. Decreasing the average alkalinity of the effluents from 1.5 to .8 and the average pH concentration factor from 6.8 to 6.2 reduced in twenty-four hours the average color of the effluents of six filters from 17 to 4, and this improvement of the filters by acid treatment seems to last

TABLE NO. 4.

EFFECT OF ADDING SULPHURIC ACID TO WATER APPLIED TO FILTERS. RESULTS BEFORE AND EIGHTEEN HOURS AFTER APPLICATION OF THE ACID.

Filter No.	Color.		Alkalinity.		pH.	
	Before.	After.	Before.	After.	Before.	After.
488	28	9	1.5	1.0	6.6	6.2
494	21	4	1.3	0.5	6.8	5.9
512	15	4	1.4	1.0	7.2	6.6
514	8	2	1.7	0.8	7.2	6.4
528	16	5	1.1	0.5	6.1	5.9
533	11	2	1.7	1.2	7.0	6.3
Average..	17	4	1.5	0.8	6.8	6.2

through a considerable period of operation. There is one curious fact in regard to this, moreover, and that is the improvement occurs without any relation to the absolute pH of the effluents. For instance, Filter No. 528, giving an effluent with a color of 16 and this effluent having a pH factor of 6.1 was by the acid treatment, improved so that the color of the effluent was 5 and the pH 5.9. The effluent from Filter No. 533 having an average color of 11 and a pH of 7.0 was by the acid treatment improved so that the color of its effluent averaged 2, but the resulting pH was 6.3 or .2 greater than the effluent from Filter No. 528 before the acid treatment. Other illustrations of this can be found in the table; that is, as I stated, the lowering of the pH seemed to produce this improvement in color removal efficiency irrespective of the resulting pH of the effluent. It may be due to some physical action of the precipitated hydroxides in the filter brought about by this acid treatment.

Cost of Process.

The cost of chemicals necessary for loading an acre of sand filter is approximately $5 000 if either aluminum sulphate or ferric sulphate is used. If ferrous sulphate can be used, as appears to be the case and as I have stated in this paper, the cost for loading an acre would not be greater than $2 800.

On Table 5 the cost of purification is figured as if the aluminum or ferric hydroxide was actually used. As a matter of fact, however, ferric

TABLE NO. 5.

Grains of Caustic Soda Used and Aluminum or Ferric Sulphate "Used" per Gallon of Water Filtered During Entire Period of Operation.

Filter No.	Filter Started.	Caustic Soda.		Aluminum or Ferric Sulphate.	
		Grains Used.	Cost per Million Gallons.	Grains "Used."	Cost per Million Gallons.
488	May 14, 1917	.50	$2.68	.07*	$0.275
494	June 7, 1918	.51	2.73	.14	.35
496	Sept. 19, 1918	.17	.91	.05*	.196
512	Feb. 2, 1921	.27	1.45	.24	.60
514	Feb. 2, 1921	.16	.86	.50	1.25
515	Feb. 2, 1921	.46	2.46	.51	1.275
523	May 22, 1922	.16	.86	.56	1.40
525	June 8, 1922	.25	1.34	.94	2.35

* Ferric sulphate.

Figures in regard to the sulphates "used" obtained by dividing tons used in charging filters by million gallons of water filtered.

hydroxide is permanent when once precipitated in the filter, and aluminum hydroxide is also permanent unless an excessive amount of caustic is used in regenerations. The cost of regeneration depends upon the weight and cost of caustic used per million gallons of water filtered. It must vary of course to some extent as the colors of the water treated vary.

At Lawrence the caustic cost figures have varied from 90 cents to $2.60 per million gallons of water filtered. The first figure is low due to the short period of operation of the filter giving this cost, and the second figure is too high because during the first year or so of operation of this filter more caustic was used than was necessary. At Providence we did not obtain satisfactory figures in regard to costs as we were always using an excessive amount of caustic in regeneration and reducing the color removal efficiency

of the filter by hydroxide removal. I believe that under the best of conditions, however, the Providence caustic cost would have been greater than the Lawrence average costs due to the different character of the water treated at Providence. This water was during part of the year very highly colored and exceedingly turbid, full of shreds of organic matter, organisms, etc.

It seems to us after quite a number of years of trial that this process will be found of use and of value in treating fairly high-colored, soft waters. Where the water has turbidity due to organic matter in suspension, or silt, there must be above that portion of the filter sand loaded with hydroxide at least a foot of sand free from this chemical in order that the filters may be scraped from time to time without removing precipitated hydroxide.

PROCEEDINGS.

December Meeting.

Boston City Club, Boston,
Tuesday, December 11, 1923.

The President, David A. Heffernan, in the chair.

J. R. McWane, President of the McWane Cast-Iron Pipe Company, read a paper on "The Use of Small Cast-Iron Pipe for Services."

A paper on "Standardization of Fire Hose Threads" was read by J. H. Howland, Engineer, National Board of Fire Underwriters, New York City.

Adjourned.

January Meeting.

Boston City Club, Boston, Mass.,
Tuesday, January 8, 1924.

The President, David A. Heffernan, in the chair.

Mr. Charles W. Sherman. I desire to call your attention to a couple of bills before the Legislature about which you ought to know, — at least those of you from Massachusetts who are operating water works; so you ought to find out about them and advise your Senators and Representatives in the Legislature as to what you think of them. The first one, which is of the most direct interest to you, is House No. 238 — "an Act reducing the time within which unpaid water rates and charges shall become liens on real estate and relative to sending notice to owners of record." You will remember that last year's Legislature passed an act making unpaid water rates liens on property. This proposed act seeks to modify those conditions by reducing the time. It was introduced by Mr. Heffernan of Boston in behalf of the mayor. (I understand he is not related to our President.) The hearing on the bill has not been announced. This particular subject is, however, before the Joint Committee on Judiciary.

The other bill which I have is of interest to those of you who have Metropolitan water. It is House No. 84, "authorizing citizens of the Commonwealth to fish in the Sudbury Reservoir of the Metropolitan Water System." It is the same matter which comes up before the Legislature every year, and it will get by some time if we are not pretty careful. That is before the Committee on Metropolitan Affairs.

Robert E. Horton, Consulting Hydraulic Engineer, Voorheesville, N. Y., presented a paper on " Determining the Mean Rainfall on a Drainage Basin." The paper was discussed by Charles W. Sherman, Harold K. Barrows, Arthur W. Dudley, R. A. Hale, Caleb M. Saville and X. H. Goodnough.

E. Sherman Chase, Sanitary Engineer with Metcalf & Eddy of Boston, read a paper on " Copper Sulphate Treatment of Cape Pond, the Source of Water Supply for Rockport, Mass." The discussion was participated in by George C. Whipple, Henry A. Symonds and M. C. Whipple. The paper was illustrated by lantern slides.

Adjourned.

LEWIS M. BANCROFT.

Lewis Melvin Bancroft, for twenty-three years Treasurer of the New England Water Works Association, died at his home in Reading, Massachusetts, on September 5, 1923.

He was born in Reading on December 31, 1851, his ancestors having lived in that town from the time of its original settlement. He was fire protection engineer for the insurance interests for a considerable period, but became superintendent of the Reading Water Works at the time of their construction in 1893, and continued as superintendent until 1922, when the works were absorbed into a Department of Public Works.

Mr. Bancroft was connected with many enterprises in Reading, being trustee of the Savings Bank and director of the Reading Coöperative Bank. He had been selectman and representative to the General Court. The esteem in which Mr. Bancroft was held in his own town is indicated by the following quotation from the Reading *Chronicle*:

"No man ever lived, in the opinion of those who knew him, of more sterling, upright, unassailable honesty and integrity than Lewis M. Bancroft, whose unexpected though not surprising passing on Wednesday caused general sorrow. While constantly in the public eye for years as a town official, it is doubtful if he ever made an enemy, although he hewed so close to the line of exact integrity that he never compromised right to expediency. While he seldom spoke in town meeting, and then only when occasion demanded, and tersely, no one had greater weight in the town's public deliberations. Reading is benefited immeasurably by his citizenship. To him more than any one else is due the credit for Reading's water system. While Representative he secured the necessary legislation, was superintendent during construction and until failing health compelled him to relinquish the office to the Board of Public Works. If any municipal department anywhere has been less subject to criticism we have yet to hear of it. The Reading Water System is one of the big assets of the town and is a permanent monument to his memory."

Mr. Bancroft became a member of the New England Water Works Association on January 8, 1890. He was Treasurer from 1899 to 1922, inclusive; and in this position, as in every other, his record is one of efficient and unassuming service.

SAMUEL E. KILLAM.
CHARLES W. SHERMAN.
Committee.

LOUIS HENRY KNAPP.

Louis Henry Knapp was born November 17, 1848, in Buffalo, and died there on January 16, 1923. He was the second son of Stephen Lockwood and Hannah (Bates) Knapp, of New England lineage; was educated in Buffalo public schools and Union College, Schenectady, and married Elizabeth B. Williams, of New York, in 1883. They had one child, Ethelind. He became a member of the New England Water Works Association June 17, 1887.

He was graduated in 1869 as a civil engineer and commissioned first lieutenant in the United States Army, stated to be a reward of merit for being one of the first three in his class. He had taken two years of medical work, also. After several months' service at Fort Lincoln, Nebraska, he was honorably discharged from the regular army. Subsequently he entered the engineering service of the National Guard of New York and became major and engineer of the 14th Regiment, April 16, 1881, and lieutenant-colonel of the Engineers Department of the 4th Division, June 3, 1882. On November 18, 1912, he was retired with rank of colonel, after thirty-two years of service.

After leaving Nebraska, he had a short engagement with the Boston and Albany Railroad as superintendent of construction. He was the first man to use four locomotives together to clear the track of snow.

To the city of Buffalo the greater part of his professional life was devoted. From some time in 1870 to July 1, 1873, he was assistant engineer of the water department, in charge of the inlet pier and the tunnel under the Niagara River. Then he became chief engineer until 1876, when he went to Niagara Falls and Suspension Bridge, New York, as engineer of water works. From 1879 to 1882, he was city engineer. In 1882, Grover Cleveland, then Mayor, appointed him superintendent and chief engineer of the water works. In this office he remained for many years, during which time two other intake tunnels were constructed under Niagara River. A storage and distributing reservoir was built on the old Dodge farm, as described in *Engineering News*, January 10, 1891, capacity 116-000 000 gallons, cost $465 000, time of construction 5 years. This reservoir was designed by William Jarvis McAlpine and was built in excavation and embankment, without core wall.

The Buffalo hydrant was designed by Mr. Knapp in 1882, as described in the JOURNAL of the New England Water Works Association, Vol. XIV, No. 3.

In 1877–78 he had charge of building new water works for St. Catherines, Ontario. At this time he met Lord Strathconna, who was building the Canadian Pacific Railway. After finishing the water works, he surveyed 500 miles of line for the new railroad in the Canadian Northwest through country occupied by the Blackfeet Indians, who dubbed him the Chief

with the Glasses. The Indian chief gave him twelve eagle feathers for bravery and later sent him a box of furs for his fair dealing.

In 1904, he built a 5-span bridge over the Allegheny River at Onoville, New York. From 1905–20, Mr. Knapp was consulting engineer on water works, electric railways and steam railroads, mostly for bondholders.

March 4, 1874, he became a member of the American Society of Civil Engineers. He was a 32-degree Mason and joined Hugh de Payens Commandery No. 30, September 11, 1876. He was a member of the Sons of the American Revolution and the Alpha Delta Phi fraternity. November 10, 1910, he was made a trustee of the Buffalo City Cemetery. He drew plans for the chapel and supervised its construction. He beautified the course of Scajaquada Creek through the cemetery and park and beyond their limits he built, in 1921, a covered channel for the creek forming a huge sewer. This was one of his last works.

Thus from its beginning to its last days his life was closely interwoven with the development of his native city of Buffalo.

GEORGE W. FULLER.
ALFRED D. FLINN.
Committee.

HIRAM FRANCIS MILLS.

Hiram Francis Mills was born in Bangor, Maine, November 1, 1836, and died October 4, 1921. He was a son of Preserved Brayton and Jane Lunt Mills. His father was a prominent Bangor physician of the Thompsonian system and was highly esteemed in the community.

Hiram was educated in the public schools of Bangor and was fitted to enter the Rensselaer Polytechnic Institute of Troy, New York, from which he graduated with a degree of B.S. in the class of 1856. Among his classmates were other noted engineers, Joseph P. Davis, C. C. Martin, John A. Wilson, being among them. Before entering on independent professional work he resolved to have ten years of experience with the ablest engineers in the country on important works then in progress, and during this period he was associated with such eminent engineers as James P. Kirkwood, William E. Worthern, James B. Francis, Charles S. Storrow, and others. This experience was sought for with the association of such engineers without regard to salary. In pursuance of this plan he served on the Bergen tunnel of the Brooklyn Water Works with James P. Kirkwood, on water power measurements and distribution of power at Cohoes, N. Y., on water measurements and construction of mills at Lowell, Mass., and experiments at North Billerica on the Concord River in connection with the drainage of the adjacent lowlands. In this connection he came in contact with Charles S. Storrow, the eminent civil engineer, who was one of the commissioners for the state, and formed a lasting acquaintance and friendship which lead ultimately to his engagement at Lawrence, Mass.

In the year 1863 he was resident engineer at the east side of the Hoosac Tunnel and designed and built the state dam in the Deerfield River at that point. Three years later he designed a stone dam on the Penobscot River at Bangor and made plans for the development of a water supply.

About the year 1868 he opened an office in Boston for professional work, making a specialty of hydraulic engineering and difficult foundations. In 1869 he conducted a series of tests on water turbine wheels at the Wamesit Power Company canal at Lowell, Mass., to determine their efficiency in the use of water and power. These included the Swain wheel, Risdon, Leffel, Bodine and others which have been superseded by improved and a higher class of wheels.

In the year 1869 he accepted the position of chief engineer of the Essex Company at Lawrence, Mass., which had developed a large water power on the Merrimack River twenty years previous and had founded the city of Lawrence. Charles S. Storrow, who designed and developed the power and laid out the city, held the position of treasurer of the company. Mr. Mills began extensive hydraulic experiments in connection with development of methods and instruments for the daily measurement of water leased to the various mills at Lawrence.

Among his studies was one which resulted in the improvement of the Pitot tube for convenient and accurate gaging purposes and brought the instrument to a high degree of efficiency. It has never been placed on the market as a commercial venture as it was always Mr. Mills idea that the profession should receive the benefit of the results. He contributed an interesting paper on the subject relating to Piezometers to the American Academy of Arts and Sciences in April, 1878.

Between 1870 and 1885 Mr. Mills had a large outside practice in the development of hydraulic power in New England and the West and was closely connected with the construction of a number of mills including difficult foundations.

Mr. Mills went to Lowell in 1893, first as consulting engineer and later became engineer of the Proprietors of the Locks and Canals at Lowell. His particular work in addition to operating the water power was to study, plan and carry out extensive changes in the canal system and the Merrimack River in order to permit the mills owning stock in the Proprietors to use to better advantage their leased power and to increase their use of permanent and surplus power from something like 6 000 second feet to more than 8 000. These improvements usually included the design of a more uniform and smoother cross section of the canals than a larger one which was not usually possible on account of the city of Lowell and the mills being built up to the canal borders; and a deepening of the river bed below the mills brought about by successive plowings, excavating and cleaning up of the channel from year to year with the river to aid; and dikes at tactical points which became the nucleus of large beds of sediment formed by the river from the material brought down by the spring freshets. Mr.

Mills' training and taste was rather in the direction of assisting nature than trying to change the face of nature by expensive excavation. Canals and river were all confined to definite channels of ample capacity and the filling of shoal places has brought into the market a large amount of land formerly unfit for use.

In 1886 Mr. Mills was appointed a member of the newly organized Massachusetts State Board of Health and was elected by his associates chairman of its committee on water supply and sewerage and served as such until the board was legislated out of existence in 1914. His devotion to this work and to the interests of the state has seldom been equaled and never surpassed by any of the men who have served Massachusetts faithfully and well upon her state boards and commissions. He very soon organized an extensive system of water sampling and analysis of all the water supplies and of the important rivers of the Commonwealth, and a little later began extensive experiments on the purification of water and sewage. Although there was no salary attached to his position and his labors were solely for the public welfare, the excellent and earnest character of the work soon commanded the respect of the Legislature and uncommon liberal appropriation were placed at the disposal of Mr. Mills. In this way the famous Lawrence Experimental Station was developed, which, all things considered, is probably the leading laboratory in the world devoted solely to studying the purification of water and sewage. His services in connection with those of Dr. Henry P. Wolcott, chairman of the old State Board with the other members gave a fine record to the board. Deaths from typhoid for the whole state dropped from 45 per 100 000 persons to about five when the old board was abolished, while the death rate for the state as a whole from all causes dropped during the period when Mr. Mills was a member from about 20 per 100 000 to 14.90.

Among the important public enterprises with which Mr. Mills has been connected in a position of the highest responsibility may be mentioned the preliminary studies for the improved sewerage and drainage of the Boston Metropolitan district, the water supply of the Boston Metropolitan district, the damming of the Charles River and various lines of investigation in connection with the general subject of public health. None of these are more thoroughly characteristic of his public spirit, however, than the construction of the water filter at Lawrence. As long ago as 1875, when the public water works were built in this city, Mr. Mills doubted the expediency of drawing a supply from the polluted Merrimack River, for he did not believe in the theory of the rapid self-purification of running streams which was at that time held strongly in England.

As opportunity offered, he investigated the typhoid epidemics along the river and eventually became as earnest advocate of the filtration of this Merrimack River supply for domestic use. After a great deal of opposition had been overcome, an appropriation for a filter was made by the city authorities and the direction of the work was left in Mr. Mills' hands,

as he had agreed to assume all the responsibility for the design and construction, without any expense to the city. This was the most noted filter in the country for many years, and its effect on the death rate was marked.

While typhoid fever carried away far more people in Lawrence than in most cities, before the filter was put in operation, the rate has remained since then below the normal. The vital statistics of the city show that probably about sixty lives annually have been saved by this work, not to mention the many cases of sickness from which recoveries are made. In this undertaking as in many others of a public nature, Mr. Mills served entirely without compensation and often against considerable opposition, to achieve results he considered of general advantage. It is only among men who have worked under him that the importance of his labors for the profession and the public are fully recognized, which is a source of great regret to them and to his non-technical friends who are enjoying the results of his professional ability.

Mr. Mills was granted the degree of Master of Arts by Harvard University in 1889. He was elected a fellow of the American Academy of Arts and Sciences in 1877; a member of the corporation of the Massachusetts Institute of Technology in 1885, and was made an honorary member of the American Society of Civil Engineers in 1909.

He has published many professional papers, among which are the following: "Experiments Upon Piezometers Used in Hydraulic Experiments," "Water Power of The United States," "Experiments Upon Central Discharge Water Wheels," "Protection of the Town of Westfield from Future Floods," "Construction of the Pacific Mills Chimney," "The Protection of the Purity of Inland Waters," "Purification of Sewage by Applying It to Land," "Report of The State Board of Health Upon the Sewerage of the Mystic and Charles River Valleys," "A Classification of the Drinking Waters of the State," "Report of The State Board of Health on Filtration of Sewage and of Water and Chemical Precipitation of Sewage," "Purification of Sewage and of Water by Filtration," "The Filter of the Water Supply of the City of Lawrence and Its Results," also memoirs of John C. Hoadley, James B. Francis and Charles S. Storrow, leaders in the engineering world.

In the year 1917 Mr. Mills resigned his position at Lowell and removed his residence to Hingham, where he had resided since that time. He had been busy, as his health permitted in completing various hydraulic formulas, in connection with his life work. He was married on October 8, 1878, to Miss Elizabeth Worcester, who died on August 23, 1917, at Hingham, shortly after leaving Lowell. They had no children. Mr. Mills is survived by two nephews, David Mills and William Mills, the sons of his brother, James Mills, a prominent mining engineer who died many years ago.

Mr. Mills was made an honorary member of this Association on September 13, 1906. With prudent management and wise investments he had accumulated considerable property which was a revelation to his

most intimate friends. His will containing many bequests of public interest showed his usual careful consideration and thoughtfulness, and no finer expression of the ideas and lessons drawn from the will can be found than the following comment from a prominent writer in one of the leading engineering publications of New York, "Will of a Great Engineer."

WILL OF A GREAT ENGINEER.

"For reasons only too well known to most of our readers the last will and testament of an engineer is rarely of interest outside his family circle. An exception rare as to the sum total bequeathed and notable because of the beneficiaries named, the evident thought put upon their selection and the restrictions and explanations relating to the residuary estate, is afforded by the will of the late Hiram F. Mills, which has just been made public. Of the public bequests of specified amounts, two are outstanding because of their objects and conditions. A large cancer research fund bequeathed to Harvard University speaks volumes, coming as it does from a man who, through his thirty years of devoted work on the State Board of Health of Massachusetts, contributed so much to the reduction of the mortality rate in general and to the typhoid rate in particular. Some decades ago, it is conceivable that Mr. Mills might have established a fund to combat typhoid. That is no longer an urgent need. A fund to aid in the reduction of the general death rate would not have that directness of aim to be expected from an engineer like Mr. Mills. What more natural, familiar as he was with the major causes of disease, suffering, and death, still awaiting successful attack, than that Mr. Mills should select that widespread, baffling and dreaded disease, cancer?

The minor public bequests, it may be noted in passing, show discrimination. The gifts are small but they will prove helpful materially and, in some respects more important yet, because of the appreciative recognition given to these institutions in the will of a great and thoughtful engineer.

Most directly human of all the gifts, and perhaps most characteristic of the giver, is the bequest of the residue of Mr. Mills' estate for the benefit of the needy of Lawrence and Lowell, particularly the mill workers. This gift, as stated in the will, springs from Mr. Mills' half century of professional work in making the most of the water power on the Merrimack River and upbuilding the cities of Lawrence and Lowell. The gift shows an appreciation of the human as well as the mechanical factor in the growth of these communities.

Mr. Mills' bequests prove anew that a man's deeds are his best monument. They also show that a last will and testament may be the most self-revealing of the documents a man leaves among the written memorials of his life work and his deepest and most heartfelt aspirations."

ARTHUR T. SAFFORD.
RICHARD A. HALE.
Committee.

INDEX OF ADVERTISERS.

	Page
Ambursen Construction Co.	ii
Badger Meter Mfg. Co.	xv
Barbour, F. A.	ii
Barrows, H. K.	ii
Boston Pipe & Fittings Co.	xxvi
Braman, Dow & Co.	xxiv
Buffalo Meter Co.	x
Builders Iron Foundry	ix
Byers, A. M., Co.	xxii
Caldwell, Geo. A., Co.	xxiv
Central Foundry Co.	xl
Chadwick-Boston Lead Co.	xxxvii
Chapman Valve M'f'g Co.	xxx
Clark, H. W., Co.	xxiv
Coffin Valve Co.	xxviii
Commonwealth Pipe & Supply Co.	xxvi
Conard & Buzby	ii
Dixon, Jos., Crucible Co.	xxiv
Donaldson Iron Co.	xxxii
Eddy Valve Co.	xxxi
Edson Manufacturing Co.	xxxv
Fairbanks-Morse & Co.	xxiii
Fay, Spofford & Thorndike	ii
Ferullo, G., Co.	iii
Fox, John & Co.	xxxiv
Fuller & McClintock	ii
Gamon Meter Co.	xiii
Gilchrist Co., Geo. E.	xxiv
Hayes Pump and Machinery Co.	xviii
Hays M'f'g Co.	xxxix
Hazen & Whipple	ii
Hersey M'f'g Co.	iv
Houdlette, Fred. A., & Son	xxiv
Hubbard-Floyd Co.	xxvi
Hubbard, W. B. & Sons Co.	xxvi
Kennison, Karl R.	iii
Lap-Joint Impervious Pipe Co.	xxiv
Lead Lined Iron Pipe Co.	xxxiv
Leadite Co.	xxxvi
Lead-Hydro-Tite	xxxviii
Lock Joint Pipe Co.	xli
Ludlow Valve M'f'g Co.	xxix
Main, Charles T.	ii
Mazzur, F. A., & Co.	xvi
Merrimac Chemical Co.	xx
Metcalf & Eddy	ii
Michigan Pipe Co.	Back cover
Mueller, H., M'f'g Co.	xxv
National Meter Co.	xiv
National Water Main Cleaning Co.	xxi
Neptune Meter Co.	vi
Pitometer Company	xxxii
Pittsburgh Meter Co.	vii
Power Equipment Co.	xviii
Rensselaer Valve Co.	xxix
Resisto Pipe & Valve Co.	xxi
Ross Valve M'f'g Co.	xxviii
Sampson, George H., Co.	xxvi
Simplex Valve & Meter Co.	xii
Smith, The A. P., M'f'g Co.	xxxiv
Starkweather & Broadhurst	xvii
Sumner & Dunbar	xxvi
Symonds, Henry A.	iii
Thomson Meter Co.	v
Thorpe, Lewis D.	iii
Union Water Meter Co.	xi
U. S. Cast Iron Pipe and Foundry Co.	xxxiii
Wallace & Tiernan	xix
Ware Coupling and Nipple Co.	xxviii
Warren Foundry and Pipe Co.	xxxv
Weston & Sampson	ii
Wood, R. D., & Co.	xxvii
Worthington Pump and Machinery Corp.	viii

(Classified index on page xiii)

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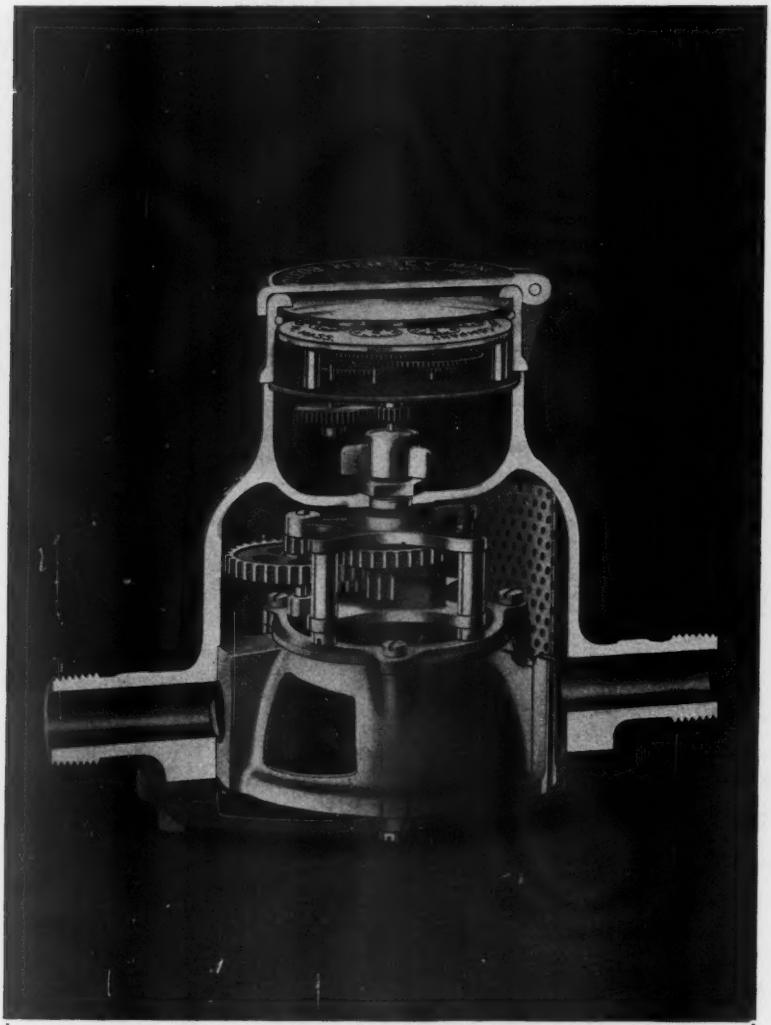

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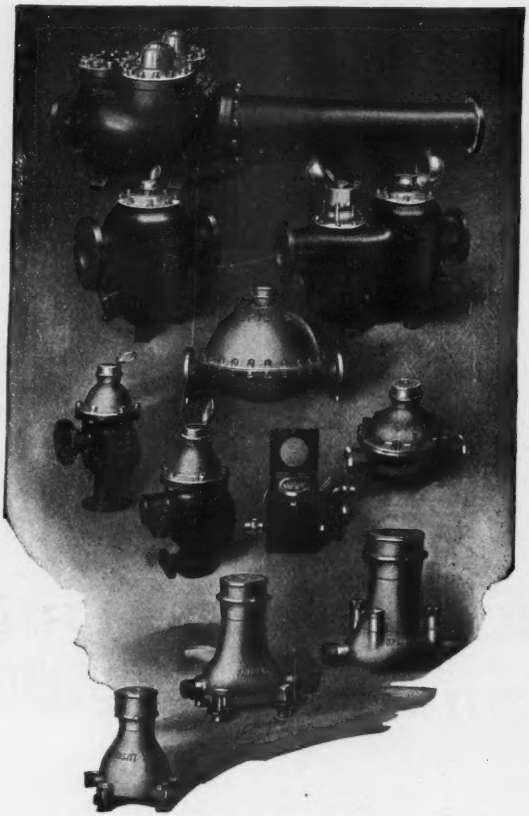

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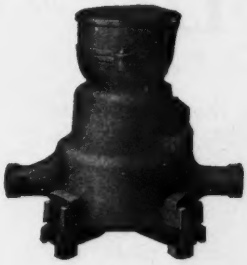

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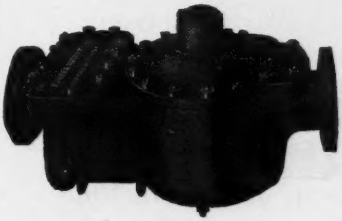

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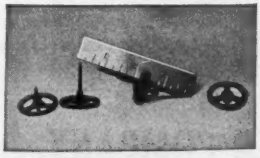
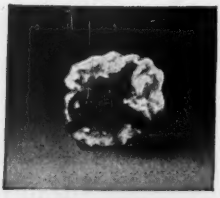
What Freezing Did Not Do

These two pictures show the intermediate gears of a Worthington Model "G" Meter. In one the train is encased in a solid block of ice, just as the parts were taken out of a completely frozen meter. After the ice melted the other photograph was made to show that no damage was done. Gears, train plate and casing, all came through the freezing totally unharmed. The only parts which suffered were four little bronze frost clamps, replaceable for a few cents. This shows some of the things that do *not* and *cannot* happen when a Worthington meter freezes. Those little frost clamps are certainly cheap insurance against the ravages of zero weather.

WORTHINGTON



Showing the gears, pinions and train plate after thawing. Not a single part has been warped or distorted in the least. The straight edge across the face of the train plate shows clearly that the original shape has not changed.



The intermediate train just after it was taken out of a standard Worthington meter. Note how completely the gearing is encased in ice.

WORTHINGTON PUMP AND MACHINERY CORPORATION

Executive Offices: 115 Broadway, New York City

Branch Offices in 24 Large Cities

Visit
The New Filtration Plant
at Cambridge

THE Cambridge plant is typically representative of the great advancement in the art of rapid sand filtration and filter plant construction.

Naturally, dependence for continuous efficiency, both in bacterial results and cost of operation, falls on *Venturi Meters* and *Venturi Filter Rate Controllers.*

Builders Iron Foundry

"Builders of the Venturi for 33 Years"

PROVIDENCE, RHODE ISLAND

AMERICAN AND NIAGARA WATER METERS

5/8 INCH NEW AMERICAN WATER METER

5/8 INCH NEW NIAGARA WATER METER

Niagara and American Meters are of the disc type. The Niagara Meter has a galvanized cast-iron outside casing; the American Meter has a bronze main casing with either a bronze base or a galvanized cast-iron base. The works in the three different casings are the same and interchangeable. Upon opening the meter at the bolted flange, each intermediate gear may be immediately removed from its bearing, the measuring chamber lifted from its seat, the strainer slipped out, or the register tried by turning the stuffing box gear. All submerged working bearings are protected against sand and sediment. The hard rubber measuring disc is reinforced with a metal plate. Purchaser has option of round reading or straight reading register indicating cubic feet, U. S. gals., imp. gals. or litres.

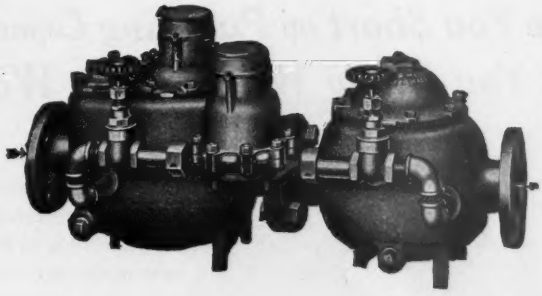
Round Reading Register

Straight Reading Register

BUFFALO METER CO.

ESTABLISHED 1892

2896 Main Street BUFFALO, N. Y.



The New Nilo

The new improved **Nilo Compound Meter** now comes equipped with a bronze body. This is a valuable addition to meter service for it eliminates the danger of corrosion and filling up due to the collecting of corrosive matter. Bronze being stronger than iron the gross weight of the meter is less, making it easier to transport, handle and install.

It also has two gate valves which allow the removal of the by-pass meter for cleaning or repairing without shutting off the line.

The controlling valve mechanism of the new **Nilo Compound Meter** is self contained and easily detached from the main body casings.

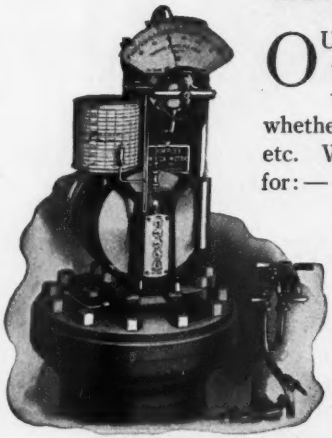
These three improvements make the **Nilo,** a long standing favorite, the unrivaled leader in its field.

UNION WATER METER CO.

Incorporated 1868

WORCESTER, MASSACHUSETTS

Is Your Pressure Low?
Are You Short on Pumping Capacity?
Do You Know Where Your Water Goes?



OUR Engineering Department will tell you how to determine your water distribution, how to find out whether or not new mains are required, etc. We can supply you with the means for: —

1. Checking up pump performance and determining slippage due to leaky plungers, defective valves, short stroking or other causes,
2. Measuring delivery of centrifugal pumps and figuring whether or not the station is up to efficiency,
3. Distributing proportionately the cost of water supplied to several districts,
4. Checking up the performance of filter beds,
5. Detecting waste or pipe leakage,
6. Recording daily amounts of sewage handled and planning future extensions, etc., etc.

THE SIMPLEX METER

VENTURI, PILOT TUBE OR ORIFICE TYPES

THE importance of exact measurements by means of a Simplex Meter cannot be over emphasized. Mr. Jeffries, Chairman of the West Chester, Pa. Water Works Committee, was able, by checking the performance of his pumping engine with a Simplex Meter, to discover that due to the imperfection and wear in the valves the slippage amounted to 300,000 gallons of water per day. This represented a daily loss of $30.00, or in other words a daily saving of $30.00, or $900.00 per month, when the valves were put in order. The Simplex Meter thus paid for itself in a month's time.

Frequent repeat orders from satisfied customers who formerly used other meters are the best evidence of merit of the Simplex Meter.

We have had many years' experience in solving water flow problems involving all sorts of conditions, and we are at all times prepared to submit general layouts involving cost and capacity, and to make explicit and complete recommendations. Write for Bulletin N26.

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Manufacturers of Meters for Water, Sewage, and Other Liquids, Rate Controllers, Automatic Air Valves, Regulating Valves, and Hydraulic Apparatus of Special Design.

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"WATCH DOG" WATER METERS

ACCURATE *DURABLE*

DISC, CURRENT AND COMPOUND TYPES

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GAMON METER COMPANY

NEWARK **NEW JERSEY**

WATER METERS

WE MAKE ALL KINDS

ROTARY, AND OSCILLATING PISTON; VELOCITY; COMPOUND; VENTURI STYLE; AND

DISC

Our Nash Disc Meters have been on the market since 1888, and are now in use by thousands, all over the country, giving the best of service. There is no better disc meter than the

NASH TYPE K

All bronze construction, substantial, accurate and durable, with straight-reading register, and enclosed intermediate.

FROST PROTECTION

Nash Type K Meters have the best of all frost protection features. The two sections of the casing are held together by a patented, breakable flange washer. In event of freezing, these washers break at the flange, allowing both casing and measuring chamber to part, thereby protecting the working parts of the meter.

Send for fully descriptive circular No. 200.

NATIONAL METER COMPANY

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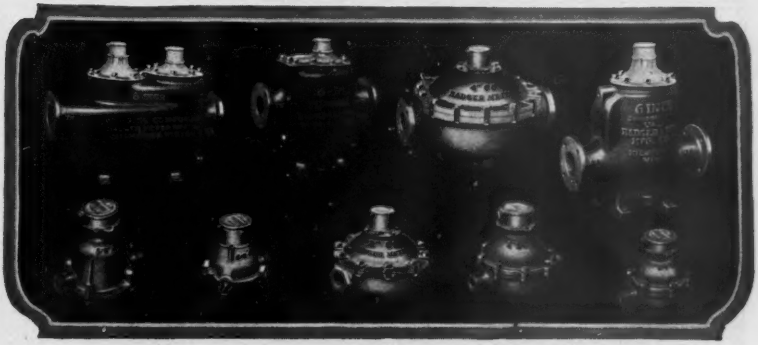
Chicago Boston Cincinnati Atlanta San Francisco Los Angeles



Make the Badger Meter your Standard for Comparison!

Just as you judge other equipment by standards — compare water meters with the Badger. Make it your **standard for comparison.** Point for point, base your rigid inspection on the outstanding construction advantages of the **Badger.** Be assured that the meter is **fit.** That it will **tick-off accurately.** That it will **register** an impartial tolling year after year. Don't accept less.

Badger Meters have long been highly reputed. Chiefly because they are built right. They are designed by practical water works engineers. All parts are readily accessible and made of such metals and materials that will not corrode, nor wear unduly. They are made for long life and service.

Let the Badger prove its dependability, accuracy and earning powers. Remember they are **faithful to the last drop.**

Write for special bulletins.

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BADGER METERS

"Faithful to the last drop"

ATTENTION
WATER WORKS
OF NEW ENGLAND

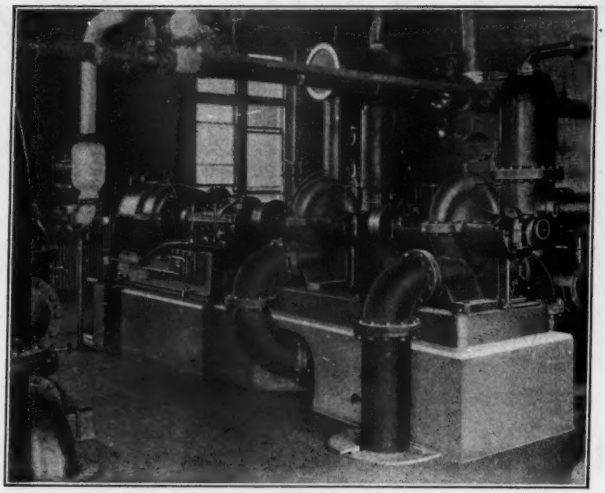
A few reasons why we solicit your inquiries on Water-Works Pumping Equipment:

First. — We make a specialty of furnishing Complete Water-Works Pumping Units of all types.

Second. — Total responsibility of installation is assumed by us.

Third. — We relieve the purchaser of every detail and turn the complete unit over to him under actual operation and with guarantee.

Fourth. — Every unit installed to date has exceeded its guarantee and has been completely accepted.



Turbine-driven Pump at the Arlington Station of the Metropolitan Water Works, Boston, Mass.

F. A. Mazzur & Co.

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Centrifugal Water Works Pumps

BUILT FOR THE LAST 70 YEARS AT BALDWINSVILLE, N. Y.

By Morris Machine Works

Reliable **Substantial** **Efficient**

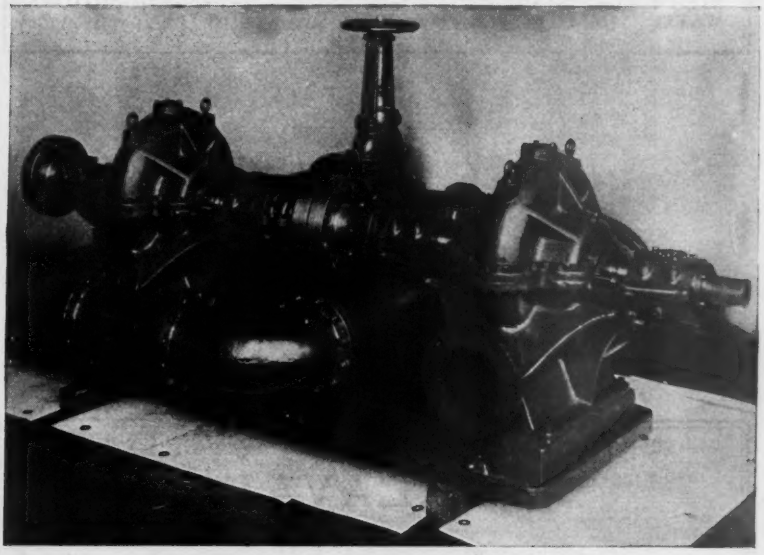

Illustration shows two 3 M.G.D. pumps just furnished as part of complete waterworks equipment to Kennebec Water District, Waterville, Me., to plans and specifications of Metcalf & Eddy, Consulting Engineers, Boston. These pumps will be driven by direct connected Motors and with Gasoline Engines for stand-by service. Arrangement permits single or series operation depending upon head, which varies greatly with capacity required.

We furnish pumps for all heads and capacities, motor turbine or engine driven, and when desired install complete units ready for operation.

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CENTRIFUGAL, POWER, STEAM PUMPS

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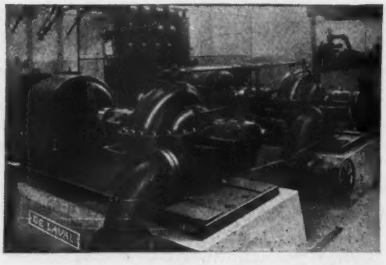

Typical Small Electrically Operated Centrifugal Municipal Water Works Installation.

POWER EQUIPMENT COMPANY
BOSTON

When The Snow Melts!

SPRING Floods will come—excessive pollution will be washed into water supplies—and increased vigilance will be necessary to prevent the spread of Water Borne Disease.

For nearly four thousand communities in North America sterilizing every drop of drinking water W & T Apparatus will stand guard against this inevitable spring time pollution.

Is your town insured against water borne disease?

"The only safe Water is a Sterilized Water"

WALLACE & TIERNAN
COMPANY, INCORPORATED
Manufacturers of Chlorine Control Apparatus
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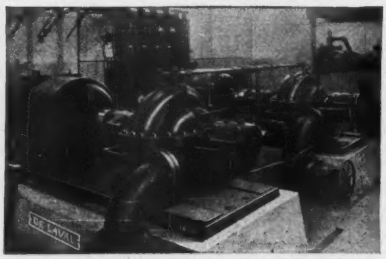

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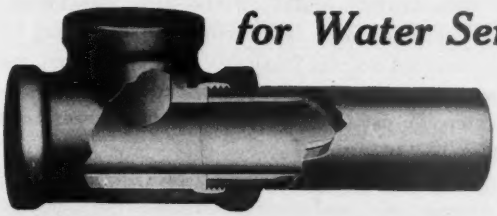

The "RESISTO" Process ensures the safest and most dependable material for water service lines. Continuous lead contact. Absolutely tight joints. No opportunity for corrosion.

Write us for details and prices.

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Resisto Pipe & Valve Company

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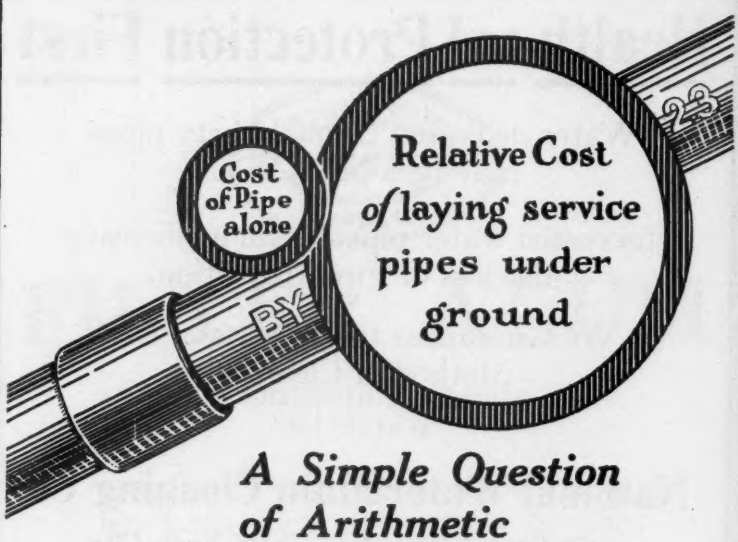

A Simple Question of Arithmetic

The cost of a length of service pipe is trifling compared with the cost of laying it. In case of failure of the pipe, the loss of water, labor of digging up and renewing a pipe, still further multiplies the installation cost.

The lasting quality of the pipe, depending on its rust-resistance, is therefore the first thing to be considered by the majority of experienced water works engineers and superintendents. Hence the extensive use of Byers pipe for underground water service.

The chemical and physical properties of genuine wrought-iron and their bearing on the question of rust-resistance, are interestingly dealt with in Byers Bulletin No. 6-A. Write to A. M. Byers Company, Pittsburgh, Pa., for a free copy.

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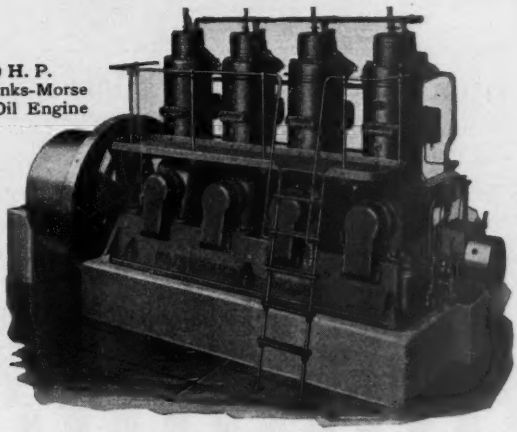
GENUINE WROUGHT IRON

Look for the Name and Year rolled in every length

Over 90,000 H. P. in use

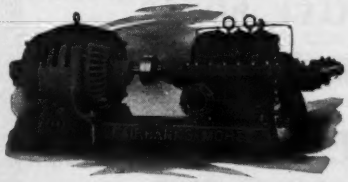
for Pumping only



200 H. P.
Fairbanks-Morse
"Y" Oil Engine

Municipal pumping station records in towns and cities throughout the United States, prove the economy of Fairbanks-Morse "Y" Oil Engines. These plants now use a total of more than 90,000 H. P. of these engines, entirely for pumping purposes.

"Y" Oil Engines use any low-grade crude or fuel oils. They start easily; full power can be had any time in five minutes. There is no expense when engine is not operating. There are no igniters, timers, wires, carburetors, hot bulbs or water injectors. Engineers like to operate them because they leave more time free for care of other equipment. Users report savings in comparison with hydro-electric and steam, of from 50 to 75 per cent.



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Fairbanks-Morse Pumps can be furnished in either centrifugal or piston types; motor, steam or oil-engine drive; either belted or direct connected. A wide range of models and sizes meets every requirement and condition.

Write us for detailed information as to the sizes and combinations of either pumps or oil-engine units that will give you the greatest economy. We will give you exact figures as to the results that may be obtained.

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Manufacturers of the well-known CLARK METER BOX, maintaining uniformity for both large and small meters. Standardization in meter installations saves you money. Everything for the Water Works.

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WROUGHT PIPE
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CEMENT LINED SERVICE PIPE
With Specially Adapted Lead-lined Fittings.
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REDUCING, REGULATING & RELIEF VALVES
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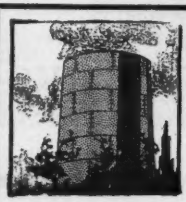

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and all other metal surfaces need the protection of
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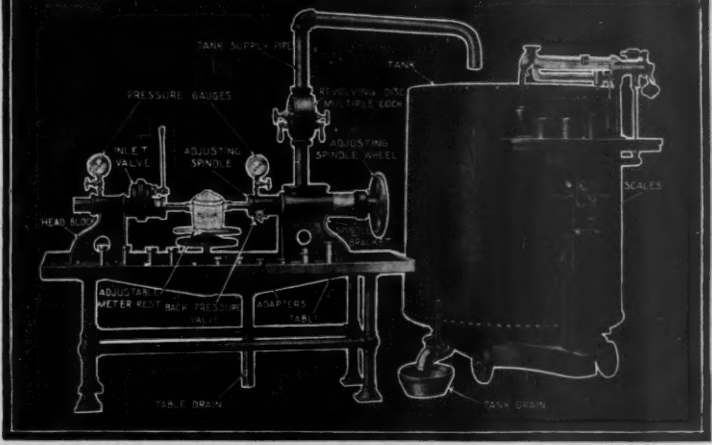

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You can make sure your Department gets full revenue for every gallon of water used—by seeing that all meters are regularly tested for accuracy—with the

Mueller Water Meter Tester

The Mueller will prove exactly what each meter does—will enable you to adjust each meter to absolute accuracy—will save more than its cost year after year. Used and endorsed by leading municipalities. The name Mueller guarantees both Quality and Service.

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Engineers, Iron Founders and Machinists

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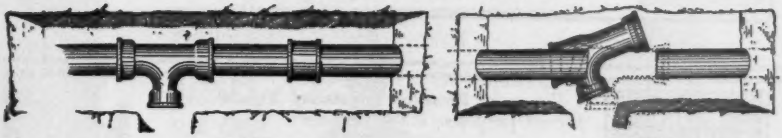
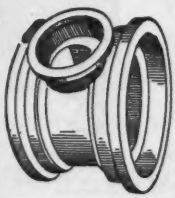
PUMPING ENGINES

Cast Iron Pipe

CUTTING-IN TEES



Old Way *Our Way*



Connections economically and easily made with one fitting. Saves sleeve, cuts, lead and unnecessary work and material.

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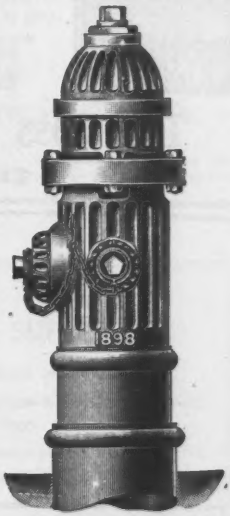
Cost of fittings reduced from 25% to 50%. Full strength. Deep bells. Convenient to handle. Sold by the piece.



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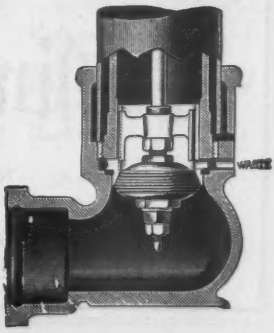
A half century of use has established their reputation as being the most economical, durable and simple hydrant.

Number in use exceeds total of all other makes combined.



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R. D. WOOD & CO. STANDARD
DOUBLE DISK
ANTI-FRICTION
EXTRA WEIGHT, FINISH
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Ware Coupling & Nipple Co.

WARE, MASS.

Grip Pipe Fittings and Brass Nipples

WATER
FILTERS
REGULATORS
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We make **Pressure Regulating Valves** for all purposes, steam or water.

Our **Feed-Water Filter** will keep oil out of your boiler.

We can interest you if you use a condenser.

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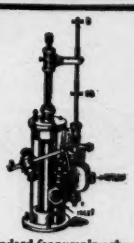

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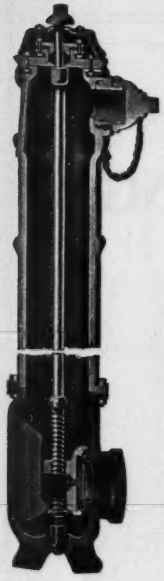

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MANUFACTURERS OF

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This hydrant is anti-freezing, because when the drainage is good no water is left in it to freeze.

The drip is directly in the bottom of the hydrant and drains perfectly. It is protected by its valve, which never leaves its socket and cannot be clogged.

DOUBLE AND SINGLE GATE **VALVES,**

ALSO CHECK **VALVES,**

FOOT **VALVES.**

FIRE **HYDRANTS.**

YARD, WASH, AND FLUSH **HYDRANTS.**

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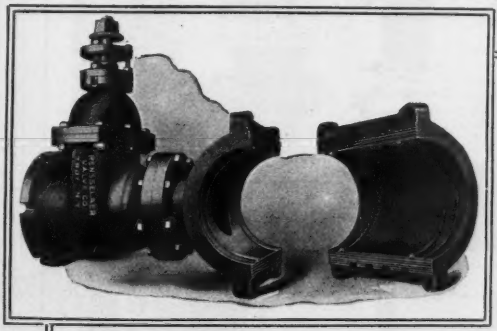

Rensselaer Tapping Sleeves

were brought out to meet a growing demand for a "BETTER" Tapping Sleeve. They were designed to suit conditions met in actual practice.

They can be used with all standard makes of Tapping Machines.

POINTS OF
SUPERIORITY

Have two lead rings.

Require less lead for calking.

Iron washers not required.

Bolting flanges are made full length of sleeve, giving great strength.

Have greater thickness of hubs on ends.

Easier to center on pipe before pouring lead.

Cost less to install.

Book No. 11 gives full information.

Ask for prices.

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RENSSELAER VALVE COMPANY, TROY, N.Y.

THE CHAPMAN VALVE MANUFACTURING COMPANY

Manufacturers of

Gate Valves Drip Valves

Gate Fire Hydrants

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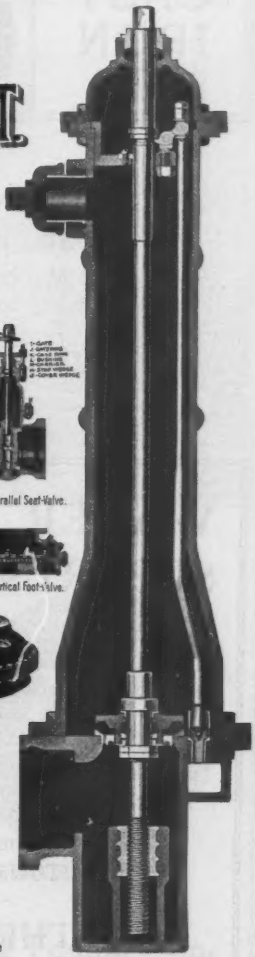
WATERFORD, NEW YORK U.S.A.

PATENT RUBBER FACED

EDDY FIRE HYDRANT



Section of Taper Seat-Valve.

Hydrant with Independent Nozzle Cut-off.

Section of Parallel Seat-Valve.

Section of Vertical Check-Valve.

Section of Vertical Foot-Valve.

Horizontal Check-Valve.

Vertical Foot-Valve.

Hydrant with Water Crane Attachment.

Indicator Post.

Water Crane with Automatic Drip Valve.

All Goods made by the EDDY VALVE COMPANY are manufactured exclusively at WATERFORD, N.Y., U.S.A.

VALVE OPEN—DRIP CLOSED.

VALVE CLOSED—DRIP OPEN.

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DONALDSON IRON CO.,

MANUFACTURERS
...OF...

CAST IRON PIPE

...AND ...

Special Castings for Water and Gas.
Also Flange Pipe, Street Castings,
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EMAUS,
LEHIGH COUNTY, PA.

WATER WORKS SUPERINTENDENTS!

Is your per capita consumption too high?

Do you account for less than 85% of your pumpage?

Are you receiving full revenue from your manufacturing consumers?

If any of these problems confront you, write us at once, without obligation, and let us tell you about our PITOMETER SURVEYS.

THE PITOMETER COMPANY
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— *" American Notes " by Charles Dickens.*

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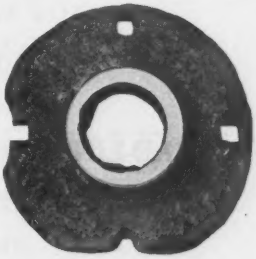
In 1842 Charles Dickens wrote the above description of the Philadelphia Water Works. At that time cast iron pipe, in Philadelphia, had been delivering water faithfully for 25 years. Nor was its usefulness nearly over. In 1915 the pipe illustrated was removed, though still in good condition after almost a century of service, to make way for a line of larger capacity.

Write to the Publicity Department, Burlington, for booklet "98 Years of Excellent Service."



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United States Cast Iron Pipe & Foundry Co.

***General Office:* Burlington, New Jersey**

THE A. P. SMITH MANUFACTURING CO.

EAST ORANGE, N. J.

Manufacturers of

Tapping Machines, Fire Hydrants, Water Gates,

Economic Lead Furnaces,

Corporation and Curb Cocks, Brass and Aluminum Castings.

Also General Supplies for Water and Gas Works.

Write for Catalogue.

NICHOLAS ENGEL

JOHN FOX & CO.

CAST IRON

WATER & GAS PIPES

FLANGE PIPE

Special Castings, Fire Hydrants, Valves

General Foundry and Machine Work

WOOLWORTH BUILDING 233 BROADWAY

NEW YORK CITY

HIGHEST AWARD, GOLD MEDAL,
ST. LOUIS EXPOSITION, 1904

Over 100 Water Departments

use

LEAD-LINED IRON and TIN-LINED PIPES

for their service connections

MANUFACTURED BY

Lead Lined Iron Pipe Company

Wakefield Mass.

Edson Manufacturing Corp.

375 BROADWAY, BOSTON, MASS.

On the Top

NO. 1 AIR-COOLED GASOLINE ENGINE
Can be attached to any Diaphragm Pump

This Engine is built for Diaphragm Pumps by the makers and inventors of the Pump.

Fifty years' experience is behind it — reliable as the original pump you have known for years.

Now in use by many of the Water Departments.

Mounted on Skid or 4-Wheel Hand Truck, also either Battery or Magneto Ignition.

Warren Foundry and Pipe Co.

(Formerly Warren Foundry and Machine Co.)

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Bell and Spigot Flanged Pipe

Special Castings

Flexible Joint Pipe Cylinders

Water Gas Sewers Culverts

Works, Phillipsburg, N. J.

LARGE STOCK ENABLES US TO MAKE QUICK SHIPMENTS

Twelve Reasons Why

YOU SHOULD USE

Trade "LEADITE" Mark

Registered U. S. Patent Office

FOR

Jointing Water Mains

1. **DURABILITY.** Leadite joints increase in strength with age.
2. **NO CAULKING.** Leadite joints require no caulking, because the Leadite adheres to the pipe, making a water-tight bond.
3. **COMPARATIVE QUANTITIES.** One ton of Leadite is equivalent to four tons of lead.
4. **LABOR SAVING.** Saves caulking charges and digging of large bell-holes, and reduces the cost of trench pumping to the minimum.
5. **COST.** Its use saves 50 to 65 per cent. over lead, owing to the saving effected in material and labor.
6. **TOOLS.** As no caulking is required, fewer tools are needed.
7. **TRANSPORTATION.** Considerable freight charges are saved because Leadite is lighter than lead.
8. **HAULING.** Saves hauling expense on the work because you move only one fourth the weight of jointing material.
9. **FUEL.** Saves fuel because you melt only one ton of material instead of four, and not as much heat is required either.
10. **DELIVERY.** We can make prompt shipments.
11. **DAMAGE SUITS.** Claims for damages caused by joints blowing out are prevented because Leadite joints will not blow out under any pressure.
12. **USERS.** Progressive water works all over the country use Leadite.

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LAND TITLE BUILDING PHILADELPHIA

Chadwick-Boston Lead Co.

162 Congress St., Boston

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The Celebrated "ULCO"

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(Every Atom Pure Lead)

For calking pipe joints under the most difficult conditions.

For overhead joints, or in wet places where the use of molten lead is not only impracticable but dangerous — **LEAD WOOL** may be used to advantage.

It makes an absolutely tight joint which will withstand the highest pressure, yet be sufficiently elastic to allow considerable sagging or settling of the pipe without danger of a leak.

As compared with the poured joint, in this respect, the superiority of **LEAD WOOL** is apparent.

Manufacturers of

Lead Pipe, Tin Lined Lead Pipe, Pure Block-Tin Pipe, Solder, White Lead and Red Lead.

HIGHEST QUALITY SOFT BRANDS

PIG LEAD

TEN CENTS PER POUND PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND TEN CENTS

Join Your Water Mains

WITH

LEAD-HYDRO-TITE

TRADEMARK

AND CUT YOUR EXPENSES

50% FIRST COST

50% SMALLER BELL HOLES

75% HANDLING

100% CALKING

OUR PROPOSAL

That the prospective users purchase a trial lot of 1 to 5 bags, and that this be used under the direction and according to the instructions of our representative.

If, after a trial, the purchaser is not satisfied with the material, any surplus not in use to be returned to the Lead-Hydro-Tite Company, and no charge to be made for any of the Lead-Hydro-Tite furnished.

Owned, Manufactured and Sold by New Englanders

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PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND PRICE TEN CENTS PER POUND

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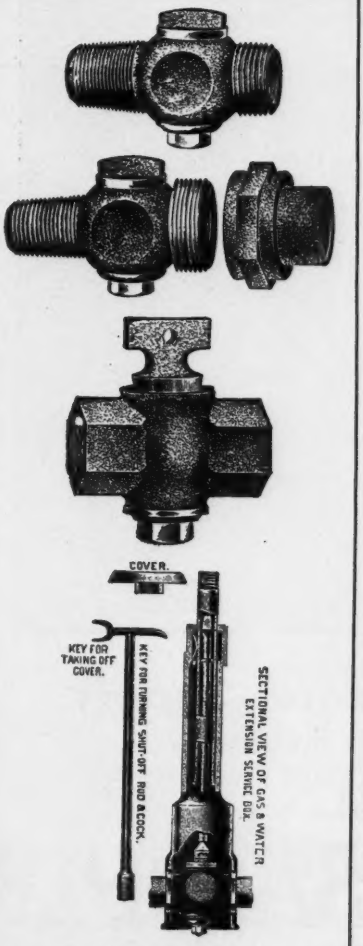

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We have them to meet every requirement

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Extension Service Boxes of proven advantages

Let us send at our risk, on 30 days' trial, a

Payne's Patent Tapping Machine

which is recognized to be the most easily operated owing to the few working parts.

Hays Mfg. Co., ERIE PENNSYLVANIA

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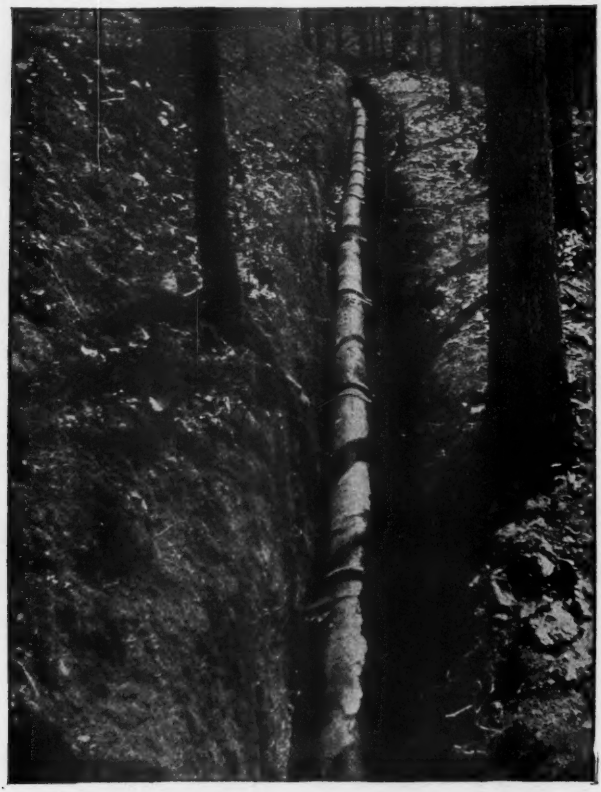

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giving perfect service
Biltmore, N. C.

UNIVERSAL CAST IRON PIPE

no packing no calking no bell holes

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A Test that is a Test

Size of pipe: 66 inches. Length of pipe line: 10 miles. Temperature at time of test: 20 degrees below zero. And the facts and results of the test were as follows: Leakage for entire ten miles in twenty-four hours, 40 000 Imperial gallons. Repairs necessary, ***none.*** Maintenance during three years, ***none.*** Capacity of pipe line, 50 000 000 Imperial gallons a day. Location, Winnipeg. We'd gladly give further facts. Write us.

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Lock Joint Pipe Company
Ampere, N. J.
Pressure, Sewer, Culvert and Subaqueous Pipe

CLASSIFIED DIRECTORY OF ADVERTISEMENTS.

PAGE.

BOILER PLANTS, FEED WATER HEATERS AND CONDENSERS.

Mazzur, F. A., & Co. . . . xvi
Starkweather & Broadhurst . . . xvii

BRASS GOODS.

Braman, Dow & Co. . . . xxiv
Caldwell, Geo. A., Co. . . . xxiv
Gilchrist Co., Geo. E. . . . xxiv
Hays M'f'g Co. . . . xxxix
Mueller, H., M'f'g Co. . . . xxv
The A. P. Smith M'f'g Co. . . . xxxiv
Union Water Meter Co. . . . xi
Ware Coupling Co. . . . xxviii

CAST-IRON PIPE AND SPECIALS.

Builders Iron Foundry . . . ix
Central Foundry Co. . . . xl
Commonwealth Pipe & Supply Co. . . . xxvi
Donaldson Iron Co. . . . xxxii
Fox, John, & Co. . . . xxxiv
Houdlette, Fred A., & Son . . . xxiv
U. S. Cast Iron Pipe and Foundry Co. . . . xxxiii
Warren Foundry and Pipe Co. . . . xxxv
Wood, R. D., & Co. . . . xxvii

CHEMICALS FOR WATER PURIFICATION.

Merrimac Chemical Co. . . . xx

CHLORINE GAS AND APPLIANCES.

Wallace & Tiernan . . . xix

CLEANING WATER MAINS.

National Water Main Cleaning Co. . . . xxi

CONSTRUCTION EQUIPMENT.

Hubbard-Floyd Co. . . . xxvi
Sampson, George H., Co. . . . xxvi

CONTRACTORS.

Ferullo, G., Co. . . . iii

ENGINEERS.

Ambursen Construction Co. . . . ii
Barbour, F. A. . . . ii
Barrows, H. K. . . . ii
Conard & Buzby . . . ii
Fay, Spofford & Thorndike . . . ii
Fuller & McClintock . . . ii
Hazen & Whipple . . . ii
Kennison, Karl R. . . . iii
Main, Charles T. . . . ii
Metcalf & Eddy . . . ii
Symonds, Henry A. . . . iii
Thorpe, Lewis D. . . . iii
Weston & Sampson . . . ii

ERECTORS, WATER WORKS AND POWER MACHINERY.

Hayes Pump and Machinery Co. . . . xviii
Mazzur, F. A., Co. . . . xvi
Power Equipment Co. . . . xviii
Starkweather & Broadhurst . . . xvii

FILTERS AND WATER-SOFTENING PLANTS.

Ross Valve M'f'g Co. . . . xxviii

FURNACES, ETC.

Mueller, H., M'f'g Co. . . . xxv
The A. P. Smith M'f'g Co. . . . xxxiv
The Leadite Co. . . . xxxvi

GATES, VALVES, AND HYDRANTS.

Chapman Valve Mfg. Co. . . . xxx
Coffin Valve Co. . . . xxviii
Eddy Valve Co. . . . xxxi
Fox, John, & Co. . . . xxxiv
Ludlow Valve M'f'g Co. . . . xxix
Rensselaer Valve Co. . . . xxix
Ross Valve M'f'g Co. . . . xxviii
Simplex Valve & Meter Co. . . . xii
The A. P. Smith M'f'g Co. . . . xxxiv
Wood, R. D. & Co. . . . xxvii

GRIP PIPE FITTINGS.

Ware Coupling Co. . . . xxviii

CLASSIFIED DIRECTORY OF ADVERTISEMENTS (*Continued*).

INSPECTION OF MATERIALS.
Conard & Buzby ii

LEAD AND LEAD PIPE.
Chadwick-Boston Lead Co. xxxvii
Lead-Lined Iron Pipe Co. xxxiv
Resisto Pipe & Valve Co. xxi

LEADITE.
The Leadite Co. xxxvi

METERS.
Badger Meter M'f'g Co. xv
Buffalo Meter Co. x
Builders Iron Foundry ix
Gamon Meter Co. xiii
Hersey M'f'g Co. iv
National Meter Co. xiv
Neptune Meter Co. vi
Pitometer Co. xxxii
Pittsburgh Meter Co. vii
Simplex Valve & Meter Co. xii
Thomson Meter Co. v
Union Water Meter Co. xi
Worthington Pump and Machinery Corp. viii

METER BOXES.
Clark, H. W., Co. xxiv
Hersey M'f'g Co. iv

OIL, GREASE, ETC.
Dixon, Jos., Crucible Co. xxiv

PIPE JOINTS.
Lead-Hydro-Tite, F. A. Houdlette & Son xxxviii
The Leadite Co. xxxvi

PRESSURE REGULATORS.
Mueller, H., M'f'g Co. xxv
Ross Valve M'f'g Co. xxviii
Union Water Meter Co. xi

PUMPS AND PUMPING ENGINES.
Builders Iron Foundry ix
Edson Manufacturing Co. xxxv
Fairbanks-Morse & Co. xxiii
Hayes Pump and Machinery Co. xviii
Mazzur, F. A., Co. xvi
National Meter Co. xiv
Power Equipment Co. xviii
Starkweather & Broadhurst xvii
Wood, R. D., & Co. xxvii
Worthington Pump and Machinery Corp. viii

RATE CONTROLLERS.
Builders Iron Foundry ix

REINFORCED CONCRETE PIPE.
Lock Joint Pipe Co. xli

SUPERHEATERS, STOKERS, STACKS, FILTERS.
Starkweather & Broadhurst xvii

SUPPLIES AND TOOLS.
Hays M'f'g Co. xxxix
Hubbard-Floyd Co. xxvi
Hubbard, W. B., & Sons Co. xxvi
Leadite Co. xxxvi
Mueller, H., M'f'g Co. xxv
Smith, The A. P., M'f'g Co. xxxiv

TAPPING MACHINES.
Hays M'f'g Co. xxxix
Mueller, H., M'f'g Co. xxv
The A. P. Smith M'f'g Co. xxxiv

UNIVERSAL PIPE.
Central Foundry Co. xl

WOOD PIPE.
Michigan Pipe Co. . . . . Back cover

WROUGHT IRON, STEEL AND BRASS PIPE.
Boston Pipe & Fittings Co. xxvi
Braman, Dow & Co. xxiv
Byers, A. M., Co. xxii
Commonwealth Pipe & Supply Co. xxvi
Lap-Joint Impervious Pipe Co. xxiv

New England Water Works Association,

ORGANIZED JUNE 12, 1882.

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I, the undersigned, residing at .....

..... *being desirous of admission into the New England Water Works Association, hereby make application for* .....

(resident, non-resident or associate)

..... *membership.*

I am ..... *years of age, and I have been engaged in the following named work:*

.....

.....

.....

I will conform to the requirements of membership if elected.

Signed, ...

Address, ...

Dated, ... *19*

Resident members are those residing in New England; all others are non-resident.
Entrance fee is $3.00 for both resident and non-resident members.
Annual dues are $6.00 for both resident and non-resident members.

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is a quarterly publication, containing the papers read at the meetings, together with reports of the discussions. Many of the contributions are from writers of the highest standing in their profession. It affords a convenient medium for the interchange of information and experience between the members, who are so widely separated as to find frequent meetings an impossibility. Its success has more than met the expectation of its projectors; there is a large and increasing demand for its issues, and every addition to its subscription list is a material aid in extending its field of usefulness. ALL MEMBERS OF THE ASSOCIATION RECEIVE THE JOURNAL IN PART RETURN FOR THEIR ANNUAL DUES; to all others the subscription is four dollars per annum.

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